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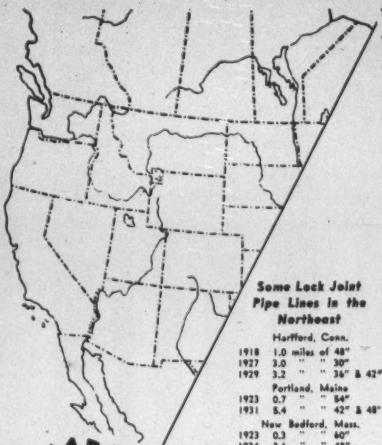

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1927 3.0 " " 30"
1929 3.2 " " 36" & 42"

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1923 0.7 " " 54"
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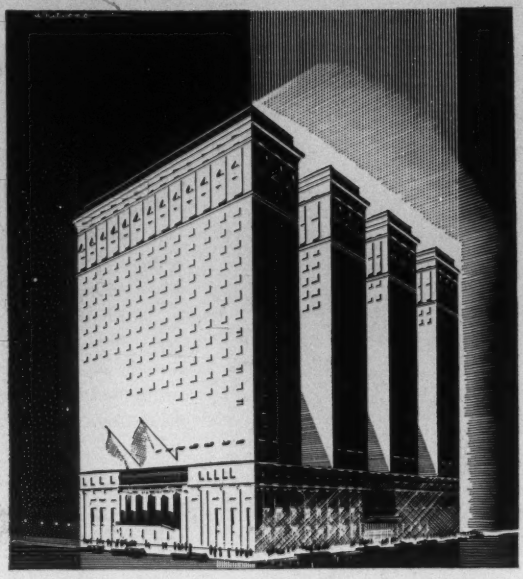

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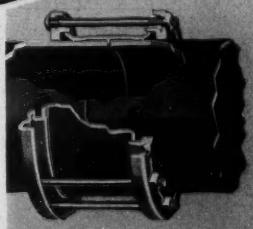

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SEEPAGE THROUGH DAMS.

BY ARTHUR CASAGRANDE.*

[Read February 23, 1937.]

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Bibliography.

*Assistant Professor of Civil Engineering, Harvard Graduate School of Engineering, Cambridge, Mass.

A. Introduction.

Until about ten years ago, the design of earth dams and dikes was based almost exclusively on empirical knowledge and consisted largely of adopting the cross-section of successful dams, with little regard to differences in character of soil and foundation conditions. At present, we are in a period in which the behavior of dams, particularly those which have failed, is analysed in the light of modern soil mechanics. The understanding and knowledge thus accumulating is being used as the basis for a more scientific approach to the design of such earth structures.

The most outstanding progress in this subject relates to the question of seepage beneath dams and dikes and to the effect of seepage on the stability of these structures. Foundation failures due to seepage, commonly known as "piping," were, for the first time, correctly explained by Terzaghi (1)* who developed what may be termed the "mechanics of piping." Later, Terzaghi (2, 3 and 4), called attention to the importance of the forces created within earth dams and concrete dams, due to the percolation of water. The practical application of this information has lagged behind our understanding of these forces partly because of theoretical difficulties of analyzing problems of seepage. It is only in recent years that substantial progress has been made in the solution of problems of seepage and ground water flow with free or open surface, of the flow through anisotropic materials, and of the conditions of flow through joint planes of different materials.

B. Darcy's Law for the Flow of Water through Soils.

The flow of water through soils, so far as it affects the question of seepage through dams, follows Darcy's empirical law, which states that *the amount of flow is directly proportional to the hydraulic gradient.* This law can be expressed either in the form:

$$\left.\begin{aligned} v_o &= k\,i \\ \text{or} \qquad Q &= k\,i\,A\,t \end{aligned}\right\} \qquad (1)$$

in which the symbols have the following meaning:

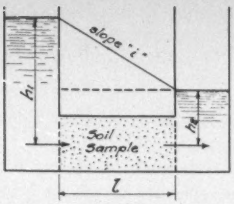
v_o = discharge velocity†
k = coefficient of permeability
i = hydraulic gradient
Q = quantity of water
A = area
t = time

In Figure 1 the meaning of Darcy's law is illustrated in simple form. A prismatic or cylindrical soil sample is exposed on the left side to a head

*Numerals refer to the bibliography at the end of this paper.

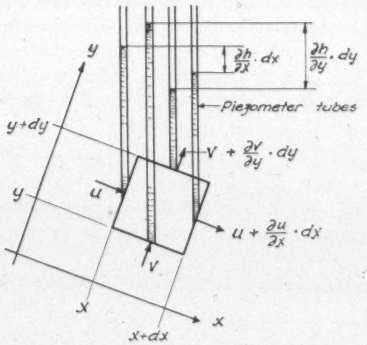
†This must not be confused with the seepage velocity $v_s = v_o \frac{1+e}{e}$, in which e = ratio of volume of voids to volume of solid matter. The average velocity through the soil is represented by the seepage velocity, while the discharge velocity determines the quantity of flow.

Darcy's Law
for Flow through Soils



Quantity of Seepage	$Q = k \cdot i \cdot A \cdot t$
Discharge Velocity	$v = k \cdot i$
Hydraulic Gradient	$i = \frac{h_1 - h_2}{l}$
Area of Sample	A
Time	t
Coefficient of Permeability	k

FIG. 1.— DARCY'S LAW FOR FLOW THROUGH SOILS.



Condition of Continuity :

$$u\,dy + v\,dx = \left(u + \frac{\partial u}{\partial x}dx\right)dy + \left(v + \frac{\partial v}{\partial y}dy\right)dx$$

or $$\frac{\partial u}{\partial x} + \frac{\partial v}{\partial y} = 0 \quad \ldots\ldots (1)$$

Darcy's Law : $u = k \cdot \frac{\partial h}{\partial x}$ & $v = k \cdot \frac{\partial h}{\partial y}$

(2) in (1) $$\frac{\partial^2 h}{\partial x^2} + \frac{\partial^2 h}{\partial y^2} = 0$$

For three dimensions $$\frac{\partial^2 h}{\partial x^2} + \frac{\partial^2 h}{\partial y^2} + \frac{\partial^2 h}{\partial z^2} = \nabla^2 h = 0$$

FIG. 2.— GENERAL DIFFERENTIAL EQUATION FOR THE FLOW OF WATER THROUGH HOMOGENEOUS ISOTROPIC SOILS.

of water h_1, and on the right side to a smaller head h_2. As a result, water will flow through the sample at a rate directly proportional to the hydraulic gradient, $i = \frac{h_1 - h_2}{l}$. If, for example, the difference in head $(h_1 - h_2)$ in Figure 1 is doubled, the quantity of seepage will also be doubled. This linear relationship suggests that the flow of water through the voids of most soils possesses the characteristics of laminar flow.

Darcy's law is frequently attacked as being incorrect. In general these attacks are based on misinterpretation of test results or improper technique of testing. In many cases they are due to loss of internal stability of the soil under the action of flowing water.

The reader may be assured that this law is valid for the study of seepage through dams.

C. General Differential Equation for the Flow of Water through Homogeneous Soil.

If water is percolating through a homogeneous mass of soil in such a manner that the voids of the soil are completely filled with water and no change in the size of the voids takes place, the quantity entering from one or several directions into a small element of volume of the soil (as shown in Figure 2) must be equal to the amount of water flowing out on the other faces of this element of volume during any given element of time. This condition, which is a statement of the fact that both water and soil are incompressible, can be expressed for the three-dimensional case by the following equation:

$$\frac{\partial u}{\partial x} + \frac{\partial v}{\partial y} + \frac{\partial w}{\partial z} = 0 \qquad (2)$$

This is known as the *equation of continuity*. (See Reference 5.) In this equation, u, v and w are the three components of the discharge velocity v_o. If $\frac{dh}{dl}$ represents the hydraulic gradient in the direction of flow and $\frac{dh}{dx}$, $\frac{dh}{dy}$ and $\frac{dh}{dz}$ are its three components, then Darcy's law:

$$v_o = k \frac{dh}{dl}$$

can also be expressed by the following equations:

$$\left.\begin{aligned} u &= k \frac{\partial h}{\partial x} \\ v &= k \frac{\partial h}{\partial y} \\ w &= k \frac{\partial h}{\partial z} \end{aligned}\right\} \qquad (3)$$

By substituting Equation (3) in Equation (2), one arrives at the general differential equation for the steady flow of water through isotropic soils. This has the form of a Laplace differential equation:

$$\frac{\partial^2 h}{\partial x^2} + \frac{\partial^2 h}{\partial y^2} + \frac{\partial^2 h}{\partial z^2} = 0 \qquad (4)$$

In our problem of seepage through dams we have to deal only with the two-dimensional case which is satisfied by the equation:

$$\frac{\partial^2 h}{\partial x^2} + \frac{\partial^2 h}{\partial y^2} = 0 \qquad (4a)$$

This equation represents *two families of curves intersecting at right angles.* (See Reference 5, p. 24 and 25.) In hydro-mechanics these curves are known respectively as the *flow lines* and the *equipotential lines* (or lines of equal head).

D. Forchheimer's Graphical Solution.

Although the general differential equation (4) has been solved only for few and simple cases of seepage, we can make use of certain geometric

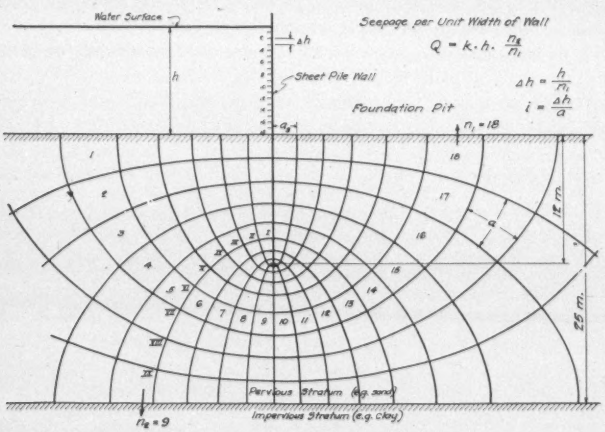


Fig. 3.—Flow Net beneath Sheet Pile Wall.

properties of flow lines and equipotential lines that permit graphical solutions for practically all two-dimensional seepage problems. This method was devised by Forchheimer (5), twenty-five years ago.

To explain this graphical method, the problem of determining the seepage beneath a sheet pile wall, shown in Figure 3 is chosen. The ground surface is a line of equal head or an equipotential line; the head being

equal to the height of water standing above the ground surface, which is h on the left side, and zero on the right side of the wall. The bottom of the pervious soil stratum is a flow line; incidentally the longest flow line. The sides of the sheet pile wall and the short width at the bottom of the wall are the shortest flow line.

If, from the infinite number of flow lines possible within the given area, we choose only a few in such a manner that the same fraction Δq of the total seepage is passing between any pair of neighboring flow lines, and, similarly, if we choose from the infinite number of possible equipotential lines only a few in such a manner that the drop in head Δh between any pair of neighboring equipotential lines is equal to a constant fraction of the total loss in head h, then the resulting "flow net," Figure 3, possesses the property that the ratio of the sides of each rectangle, bordered by two flow lines and two equipotential lines, is constant. (See Reference 5, p. 82.) If all sides of one such rectangle are equal, then the entire flow net must consist of squares. Conversely, it can be proved that if one succeeds in plotting two sets of curves so that they intersect at right angles, forming squares and fulfilling the boundary conditions, then one has solved, graphically, equation (4a) for this problem. With experience, this method can be applied successfully to the most complicated problems of seepage and ground water flow in two dimensions, including seepage with a free surface.

After having plotted a flow net that fulfills satisfactorily these necessary conditions, one can derive therefrom, by simple computations, any desired information on quantity of seepage, seepage pressures, and hydrostatic uplift. For example, the total seepage per unit of length and per unit of time is determined from the following formula, which is simple to derive from Darcy's law:

$$q = kh\frac{n_2}{n_1} \tag{5}$$

in which n_1 is the number of squares between two neighboring flow lines, and n_2 the number of squares between two neighboring equipotential lines.

The maximum hydraulic gradient on the discharge surface, which influences the safety against "piping" or "blows," is equal to:

$$i_s = \frac{\Delta h}{a_s} \tag{6}$$

in which a_s is the length of the smallest square on the discharge surface, as indicated in Figure 3, and $\Delta h = \frac{h}{n_1}$, the drop in head between two adjacent equipotential lines.

To assist the beginner in learning the graphical method, the following suggestions are made:

1. Use every opportunity to study the appearance of well-constructed flow nets; when the picture is sufficiently absorbed in your mind, try to draw the same flow net

without looking at the available solution; repeat this until you are able to sketch this flow net in a satisfactory manner.

2. Four or five flow channels are usually sufficient for the first attempts; the use of too many flow channels may distract the attention from the essential features. (For examples see Figures 18b and c.)

3. Always watch the appearance of the entire flow net. Do not try to adjust details before the entire flow net is approximately correct.

4. Frequently there are portions of a flow net in which the flow lines should be approximately straight and parallel lines. The flow channels are then about of equal width, and the squares are therefore uniform in size. By starting to plot the flow net in such an area, assuming it to consist of straight lines, one can facilitate the solution.

5. The flow net in confined areas, limited by parallel boundaries, is frequently symmetrical, consisting of curves of elliptical shape. (For example see Figure 3.)

6. The beginner usually makes the mistake of drawing too sharp transitions between straight and curved sections of flow lines or equipotential lines. Keep in mind that all transitions are smooth, of elliptical or parabolic sharpe. The size of the squares in each channel will change gradually.

7. In general, the first assumption of flow channels will not result in a flow net consisting throughout of squares. The drop in head between neighbouring equipotential lines corresponding to the arbitrary number of flow channels, will usually not be an integer of the total drop in head. Thus, where the flow net is ended, a row of rectangles will remain. For usual purposes this has no disadvantages, and the last row is taken into consideration in computations by estimating the ratio of the sides of the rectangles. If, for the sake of appearance, it is desired to resolve the entire area into squares, then it becomes necessary to change the number of flow channels, either by interpolation or by a new start. One should not attempt to force the change into squares by adjustments in the neighbouring areas, unless the necessary correction is very small.

8. Boundary conditions may introduce singularities into the flow net, which are discussed more in detail in Appendix I, e.

9. A discharge face, in contact with air, is neither a flow line nor an equipotential line. Therefore, the squares along such a boundary are incomplete. However, such a boundary must fulfill the same condition as the line of seepage regarding equal drops in head between the points where the equipotential lines intersect.

10. When constructing a flow net containing a free surface one should start by assuming the discharge face and the discharge point and then work toward the upstream face until the correct relative positions of entrance point and discharge toe are attained. Hence, the scale to which a flow net with a free surface is plotted, will not be known until a large portion of the flow net is finished. For seepage problems with a free surface it is practically impossible to construct a flow net to a predetermined scale in a reasonable length of time.

E. Seepage through Dams; General Considerations.

In almost all problems concerning seepage *beneath* sheet pile walls or *through the foundation* of a dam, *all boundary conditions are known*. However, in the *seepage through* an earth dam or dike, the *upper boundary* or uppermost flow line is *not known*, but must first be found, thus introducing a complication. This upper boundary is a free water surface and will be referred to as the *line of seepage*.

Among the available theoretical solutions for seepage with a free surface there is one case which is of particular importance in connection with our problem. It is Kozeny's solution (6) of the flow along a horizontal

impervious stratum that continues at a given point into a horizontal pervious stratum, thus representing an open horizontal discharge surface as shown in Figure 9d. In this case, all flow lines, including the line of seepage, and all equipotential lines are confocal parabolas with point A as the focus.

For the more common problem of seepage through cross-sections in which the discharge slope forms an angle with the horizontal between 0° and 180°, such as the open discharge on the downstream face of a dam or discharge into an overhanging slope of a very pervious toe, such as a rock

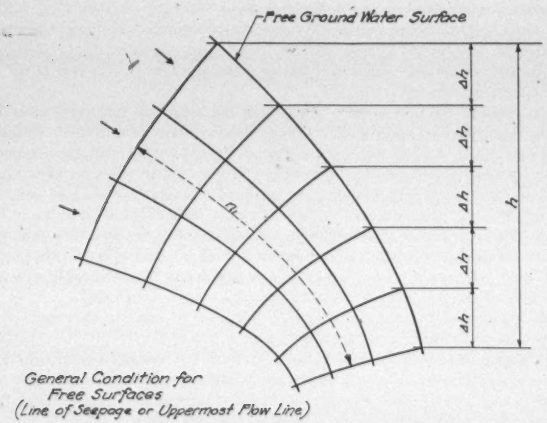


FIG. 4.— GENERAL CONDITION FOR LINE OF SEEPAGE.

fill toe, one has to use either a graphical solution based on the construction of the flow net, or some approximate mathematical solution. In either case one must introduce certain conditions that the free water surface or line, of seepage must always fulfill.

The first condition is that the elevation of the point of intersection of any equipotential line with the line of seepage represents the head along this equipotential line. If we construct a flow net consisting of squares, then it follows that all intersections of equipotential lines with the line of seepage must be equidistant in the vertical direction. These distances, illustrated in Figure 4, represent the actual drop in head $\Delta h = \frac{h}{n_1}$ between any two neighboring equipotential lines.

The second condition refers to the slope of the line of seepage at the point of intersection with any boundary, as for example at the points of entrance and discharge and at the boundary line between different soils

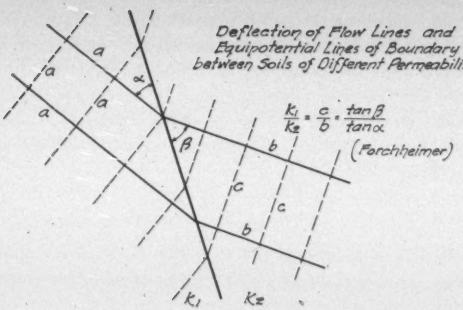


FIG. 5.— DEFLECTION OF FLOW NET AT BOUNDARY OF SOILS OF DIFFERENT PERMEABILITY.

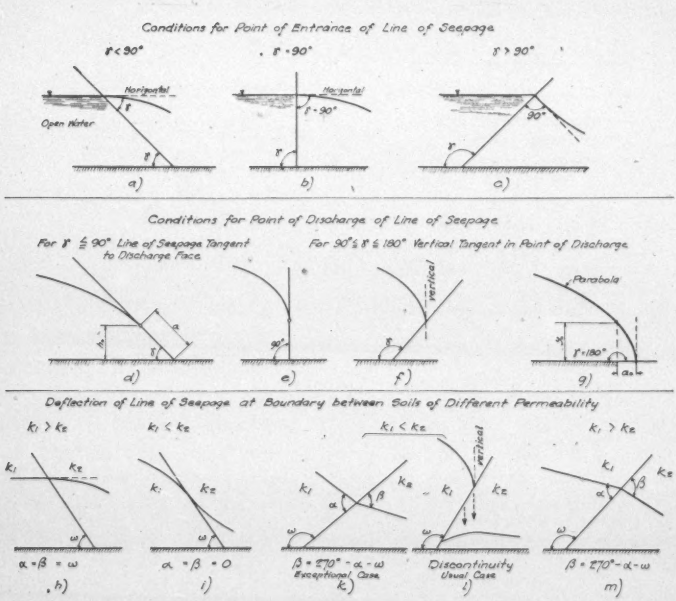


FIG. 6.— ENTRANCE, DISCHARGE AND TRANSFER CONDITIONS OF LINE OF SEEPAGE.

(See Figure 5). By considerations based on the general properties of a flow net, one can arrive at the conditions which must be fulfilled at such points of transfer. In Appendix I are assembled derivations for typical cases. If, for example, the downstream face is inclined less than or equal to 90°, one finds that the line of seepage must be tangent to that face at the discharge point. However, for all overhanging slopes, the tangent at the discharge point must be vertical. A summary of the possible combinations is assembled in Figure 6.

F. Seepage through Homogeneous Isotropic Earth Dams.

a. Approximate Solution for $\alpha < 30°$. The first approximate mathematical solution for determining the quantity of seepage and the line of seepage through a homogeneous earth section on an impervious base was

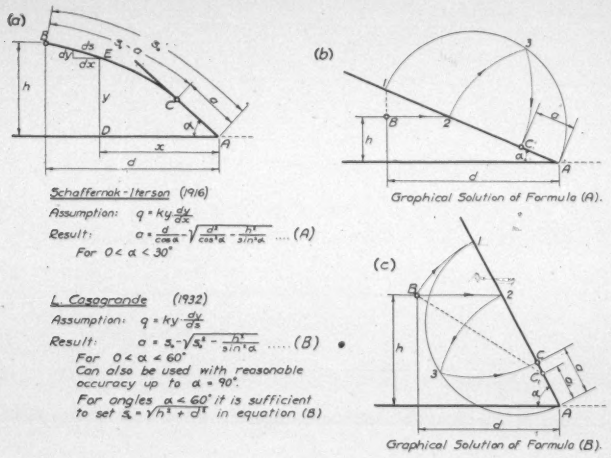

(a)
(b)
(c)
Schaffernak - Iterson (1916)
Assumption: $q = ky\frac{dy}{dx}$
Result: $a = \frac{d}{\cos\alpha} - \sqrt{\frac{d^2}{\cos^2\alpha} - \frac{h^2}{\sin^2\alpha}}$ .....(A)
For $0 < \alpha < 30°$
L. Casagrande (1932)
Assumption: $q = ky\frac{dy}{ds}$
Result: $a = s_1 - \sqrt{s_1^2 - \frac{h^2}{\sin^2\alpha}}$ .....(B)
For $0 < \alpha < 60°$
Can also be used with reasonable accuracy up to $\alpha = 90°$.
For angles $\alpha < 60°$ it is sufficient to set $s_1 = \sqrt{h^2 + d^2}$ in equation (B)
Graphical Solution of Formula (A).
Graphical Solution of Formula (B).

Fig. 7.— Graphical Determination of Discharge Point for $\alpha < 60°$.

proposed independently in 1916 by Schaffernak (7) in Austria, and Iterson (8), in Holland. It is based on Dupuit's (9) assumption* that in every point of a vertical line the hydraulic gradient is constant and equal to the slope $\frac{dy}{dx}$ of the line of seepage at its intersection with that vertical line. This assumption represents a good approximation for the average hydraulic gradient in such a vertical line providing the slope of the line of seepage is relatively flat.

*On this same assumption are based the common methods of computing ground water flow toward wells.

With this assumption and the condition that the quantity of water flowing through any cross-section per unit of time must be constant, one can derive the differential equation for the line of seepage from Figure 7a:

$$q = ky\frac{dy}{dx} \tag{7}$$

The solution of equation (7) yields the equation of a parabola. Assuming that the quantities h, d, and α, in Figure 7, are known, and with the boundary conditions $y=h$ for $x=d$, and $dy/dx=tan\ \alpha$ for $x=a\ cos\ \alpha$, or $y=a\ sin\ \alpha$, integration leads to the following formula for the distance a which determines the discharge point C of the line of seepage on the downstream face of the dam:

$$a = \frac{d}{cos\ \alpha} - \sqrt{\frac{d^2}{cos^2\ \alpha} - \frac{h^2}{sin^2\ \alpha}} \tag{8}$$

$$q = k\ a\ sin\ \alpha\ tan\ \alpha \tag{9}$$

These equations differ from their original form in the use of the distance a, instead of its vertical projection, a change which provides a common basis for all theoretical developments in this paper. A further advantage

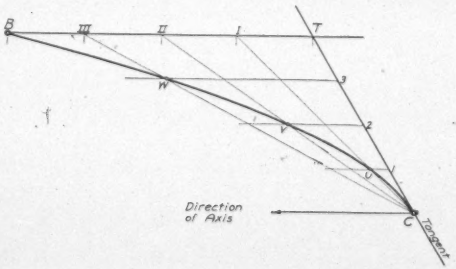
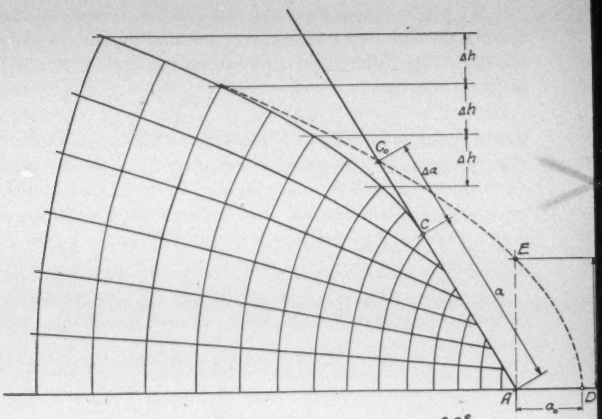


FIG. 8.— METHOD OF LOCATING POINTS ON A PARABOLA.

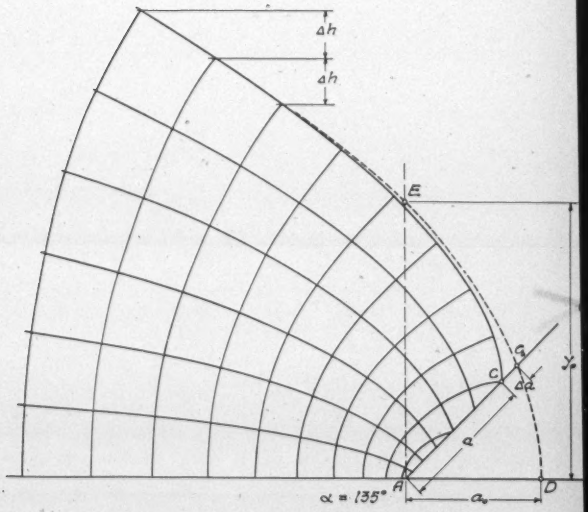
of this change resides in the possibility of determining graphically the distance a by means of a simple construction* which is shown in Figure 7b. The ordinate through the known point B of the line of seepage is extended to its intersection *1* with the discharge slope, and a semi-circle drawn through the points *1* and A, with its center on the discharge slope. Then a horizontal line through B is intersected with the discharge slope in point *2*, and the distance *2*-A projected onto the circle, yielding point *3*. The final step is to project the distance *1*-*3* onto the discharge slope. This yields the desired discharge point C. The proof for the validity of this method is readily found by comparing this construction with equation (8) and need not be discussed in detail.

*This construction is a simplification of another method which was proposed in References 10 and 11.



a) $\alpha = 60°$

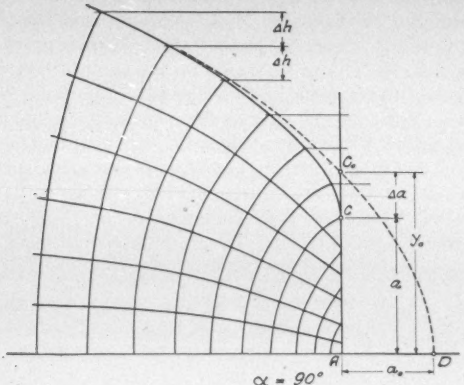
$\frac{\Delta a}{a+\Delta a} = 0.32$



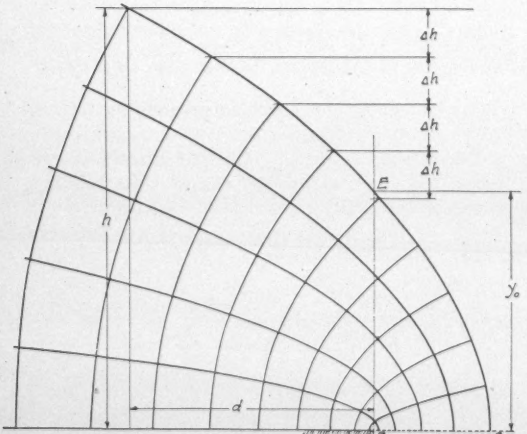
c) $\alpha = 135°$

$\frac{\Delta a}{a+\Delta a} = 0.14$

FIG. 9.— COMPARISON BETWEEN BASIC PARABO



b) $\alpha = 90°$ $\quad \frac{\Delta a}{a + \Delta a} = 0.26$ or $a \sim \frac{3}{4} y_0$



Equation of Line of Seepage $\quad x = \frac{y^2 - y_0^2}{2 y_0}$ $\quad a_0 = \frac{y_0}{2} = \frac{l}{2}\left(\sqrt{d^2 + h^2} - d\right)$

$q = 2 \cdot k \cdot a_0 = k \cdot y_0$

d)

AND FLOW NETS FOR VARIOUS DISCHARGE SLOPES.

For the solution of many problems it is sufficient to know the discharge point of the line of seepage. If it is desired to draw the entire line of seepage, from the known point B to the discharge point C, one can make use of the graphical method, shown in Figure 8, for the rapid construction of any number of points on a parabola for which are known two points, the tangent to the parabola at one of these points, and the direction of the axis.

Through point B, Figure 8, one draws a line parallel to the axis and determines its intersection T with the tangent. Then one divides the distances B-T and C-T into an arbitrary number of equal parts, such as four parts. Points I, II and III are then connected with point C, and through points *1*, *2* and *3* one draws lines parallel to the axis. The points where the lines through *1* and I, *2* and II, etc., intersect, are points of the parabola.

b. Approximate Solution for $\alpha > 30°$. The approximate solution by means of equation (8), or the corresponding graphical method shown in Figure 7b, gives satisfactory results for slopes of $\alpha < 30°$. For steeper slopes the deviation from the correct values increases rapidly beyond tolerable limits.

The causes for this deviation become apparent from a study of the flow net for a slope of $\alpha = 60°$, shown in Figure 9a. One can see that in the vicinity of the discharge point the size of the squares along the vertical line through the discharge point decreases only slightly towards the base. The average hydraulic gradient along this vertical line is larger than the hydraulic gradient $\frac{dy}{ds}$ along the line of seepage by less than 10 per cent. However, the sine of 60°, which is the true hydraulic gradient for the line of seepage at the discharge point, is only about one-half of the tangent of 60° used according to Dupuit's assumption. Hence the seepage can be analysed with a satisfactory degree of accuracy by means of the following equation:

$$q = ky\frac{dy}{ds} \tag{10}$$

This improvement was proposed by Leo Casagrande (10).

The difference between the use of the tangent and the sine of the slope of the line of seepage is best illustrated by the following numerical comparison for various angles:

Slope	*tan*	*sin*
30°	0.577	0.500
60°	1.732	0.866
90°	∞	1.000

Hence, for slopes $< 30°$ both methods may be used for practical purposes with equal advantage. For slopes $> 30°$ the deviation by using $\frac{dy}{dx}$ becomes

intolerably large, while the use of $\frac{dy}{ds}$ is very satisfactory for slopes up to 60°; and if deviations of 25 per cent. are permitted, it may even be used up to 90°, that is, for a vertical discharge face.

Gilboy (12) succeeded in finding an implicit solution of equation (10) which is recommended where greater accuracy is required than can be obtained by means of the graphical solution. The errors involved in the position of the discharge point, as obtained by one or the other method from equation (10), were investigated by G. P. Reyntjiens (13).

Using the symbols shown in Figure 7a, and assuming that in each vertical the hydraulic gradient is equal to $\frac{dy}{ds}$, equation (10) is the differential equation for the line of seepage. The solution of this equation cannot readily be expressed by rectangular coördinates x and y. (See References 12 and 13.) However, the use of s and s_o, measured along the line of seepage, does not represent any practical difficulty in the actual application of this method. The quantity a, which determines the discharge point for the line of seepage, is found by a simple integration:

$$qs = \frac{ky^2}{2} + \text{constant}$$

Boundary Conditions $\begin{cases} s = a,\ y = a\ sin\ \alpha,\ q = ka\ sin^2\alpha \\ s = s_o,\ y = h \end{cases}$

$$a = s_o - \sqrt{s_o^2 - \frac{h^2}{sin^2\ \alpha}} \qquad (11)$$

$$q = k\ a\ sin^2\ \alpha \qquad (12)$$

Again, the quantities employed in these equations differ from the original form as presented in References 10 and 11 to permit a simple graphical solution. This graphical solution of equation (11) is illustrated in Figure 7c, and can be easily verified. It requires first an assumption for the discharge point. The length $(s_o - a)$ is simply taken equal to the straight line from B to C_1, shown as a dotted line in Figure 7c. The slight error which is introduced when $(s_o - a)$ is replaced by a straight line has a negligible effect on the positions of the discharge point. In fact, for slopes $\alpha \leq 60°$, it is entirely tolerable to replace the length s_o by the straight distance from $\overline{AB} = \sqrt{h^2 + d^2}$, thus eliminating trial constructions. The construction is very similar to that shown in Figure 7b, except that point *1* is found by rotating distance $\overline{C_1B}$, or $\overline{AB}$, around point A.

If deviations up to 25 per cent. are permitted, the simplified value $s_o = \sqrt{h^2 + d^2} = \overline{AB}$ may be used also for slopes up to 90°. For a vertical slope the formula for a is reduced to the following simple form:

$$a = \sqrt{h^2 + d^2} - d \qquad (13)$$

In other words, for a vertical discharge face the height of the discharge

point for the line of seepage can be approximated by the difference between the distance $\overline{AB} = \sqrt{h^2+d^2}$ and its horizontal projection d.

c. Solution for a Horizontal Discharge Surface ($\alpha = 180°$). In 1931, Professor Kozeny (6) published a rigorous solution for the two-dimensional problem of ground water flow over a horizontal impervious surface which continues at a given point into a horizontal discharge face, as shown in Figure 9d. Kozeny's theoretical solution yields, for the flow lines and equi-potential lines, two families of confocal parabolas, with point A, where the impervious and pervious sections meet, as the focus.

The equation for the line of seepage can be conveniently expressed in the following form:

$$x = \frac{y^2 - y_o^2}{2y_o} \tag{14}$$

in which x and y are the coördinates with the focus as origin, and y_o the ordinate at the focus $x = 0$.

If the line of seepage is determined by the coördinates d and h of one known point, then the focal distance a_o and the ordinate y_o are computed from the following equation:

$$a_o = \frac{y_o}{2} = \frac{1}{2}\left(\sqrt{d^2+h^2} - d\right) \tag{15}$$

for which a graphical solution is recommended. (See Figure 11c.) The quantity y_o is simply equal to the difference between the distance $\sqrt{d^2+h^2}$ from the given point (d, h) to the focus of the parabola, and the abscissa d. The focal distance a_o is equal to one-half the ordinate y_o.

In addition to these simple relationships it is of advantage to remember that the tangent to the line of seepage at $x = 0$ and $y = y_o$ is inclined at 45°.

The quantity of seepage per unit of width is, according to Kozeny's solution:

$$q = 2ka_o = ky_o \tag{16}$$

It is indeed fortunate that the problem of seepage with a horizontal discharge face has such a simple solution, not only because of the fact that in modern earth dam and levee design horizontal drainage blankets in the downstream section are assuming considerable importance, but also because this solution permits fairly reliable and simple estimates for the position of the line of seepage for overhanging discharge slopes.

d. Approximate Solutions for Overhanging Discharge Surfaces ($90° < \alpha < 180°$). Although the determination of the line of seepage and its point of exit for an overhanging discharge face, such as a rock fill toe, is of importance in the design of earth dams, little attention has been paid to this problem. Experimental results were published by Leo Casagrande (10 and 11), which permit a reasonably accurate determination of the line of seepage. Later, in 1933, the author checked the results of these model

tests by means of graphical solutions, of which a few typical examples are shown in Figure 9. These solutions check well with the experimental results just referred to, and are illustrated for one example in Figure 10. Such studies convinced the author that Forchheimer's graphical method for the determination of the flow net can be utilized for the solution of seepage problems with a free surface. The application of the graphical method to such problems requires considerable skill. This can be acquired only by extensive use of this method. Solutions such as those shown in Figure 9

FIG. 10.— MODEL TEST ON OTTAWA STANDARD SAND WITH OVERHANGING DISCHARGE SLOPE.

Note zone of capillary saturation above line of seepage — upper dye line.
After L. Casagrande (10).

required many hours of work; sometimes several days were spent on one case,— because of the complications that the unknown upper boundary introduces into such seepage problems.

After sufficient graphical solutions to permit a rapid determination of the line of seepage for any slopes $60° \leq \alpha \leq 180°$, had been accumulated, the author's attention was called to Kozeny's (6) theoretical solution for $\alpha = 180°$. This proved a splendid opportunity for checking the accuracy of a purely graphical solution of a seepage problem with a free water surface. Figure 9d represents the original graphical solution. The difference between this solution and the theoretical solution is not more than 3 per cent. for any point on the line of seepage. Therefore, no attempt was made to include the theoretical solution in Figure 9d. This remarkable accuracy

of the graphical method should convince critics that the method is not a plaything but has great merit and that the time spent on acquiring sufficient skill in this method is well invested.

To simplify the application of the graphical solution for very steep and overhanging discharge slopes such as are shown in Figures 9, a, b and c, these flow nets were compared with Kozeny's theoretical solution for a horizontal discharge face. For the sake of simplicity, the line of seepage for $\alpha = 180°$, which is represented by equations (14) and (15), and Figure 9d, will be referred to as the "basic parabola."

In Figure 9, the basic parabola is plotted into every case illustrated. The basic parabola and the actual line of seepage approach each other very quickly and for practical purposes may be assumed to be identical for points whose ordinates h are less than their horizontal distances from the discharge point C. By comparing the actual line of seepage for a given discharge slope with the basic parabola, we find that the intersection of this parabola with the discharge face is a distance Δa above the discharge point of the line of seepage. The ratio $C = \frac{a}{a+\Delta a}$ (see Figure 9) gradually decreases with increasing angle α. The ratio C is equal to 0.32 for $\alpha = 60°$; for a vertical surface ($\alpha = 90°$) it is 0.26; and for $\alpha = 180°$ the ratio C is, of course, equal to zero.

In order to utilize these relationships for determining the line of seepage and the discharge point for steep vertical and overhanging slopes, there has been plotted in Figure 11 the relationships between the ratio $\frac{a}{a+\Delta a}$ and the angle α. The quantity $a+\Delta a$ is found by intersecting the basic parabola with the discharge slope, an operation that can be performed either graphically or mathematically. In both cases one computes or constructs first $y_o = \sqrt{d^2+h^2}-d$, for the known or estimated starting point of the line of seepage. The graphical determination of the intersection C_o is usually preferred, since the basic parabola is needed for the determination of the line of seepage. The construction of the parabola is best performed in the manner illustrated in Figure 8. For tangent $\overline{CT}$, either the tangent at the vertex of the parabola, or the tangent under 45° at $x = 0$ and $y = y_o$ can be used.

The points on the curve representing the relation between α and $c = \frac{a}{a+\Delta a}$, in Figure 11b, are derived from the graphical solutions. Note how close to a smooth curve these points lie. This is another demonstration of the degree of accuracy that can be obtained by means of the graphical method.

The quantity c is not only a function of the angle α but it also varies somewhat with the relative position of points B_1, or B_2, and C_o (Figure 11d). The maximum variations in c, for the limits that would normally be en-

countered in earth dams, are about ± 5 per cent. The curve in Figure 11b was determined for a relatively short distance from the entrance to the discharge point of the line of seepage, in consideration of the importance of stratification in earth dams which is discussed in the next chapter.

Having plotted the basic parabola and determined the discharge point by means of the $c-\alpha$ relation, Figure 11, and knowing the tangent to the seepage line at the discharge point, it is an easy matter to draw with a fair degree of approximation the entire line of seepage, as shown in the various cases in Figure 9.

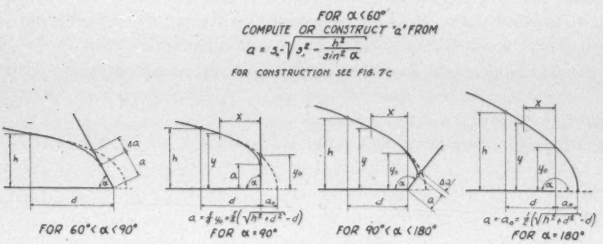
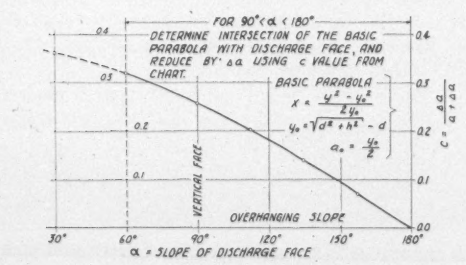

FOR α < 60°
COMPUTE OR CONSTRUCT "a" FROM
$a = s_0 - \sqrt{s_0^2 - \frac{h^2}{\sin^2 \alpha}}$
FOR CONSTRUCTION SEE FIG. 7c
FOR 60° < α < 90°
$a = \frac{1}{2}\left(\sqrt{h^2+d^2}-d\right)$
FOR α = 90°
FOR 90° < α < 180°
$a = a_0 = \frac{1}{2}\left(\sqrt{h^2+d^2}-d\right)$
FOR α = 180°

FIG. 11a


FOR 90° < α < 180°
DETERMINE INTERSECTION OF THE BASIC PARABOLA WITH DISCHARGE FACE, AND REDUCE BY Δa USING c VALUE FROM CHART.
BASIC PARABOLA
$x = \frac{y^2 - y_0^2}{2y_0}$
$y_0 = \sqrt{d^2+h^2} - d$
$a_0 = \frac{y_0}{2}$
VERTICAL FACE
OVERHANGING SLOPE
$c = \frac{\Delta a}{a}$ or $\frac{\Delta a}{\Delta}$
α = SLOPE OF DISCHARGE FACE

NOTE: POINTS ⌒ WERE FOUND BY GRAPHICAL DETERMINATION OF FLOW NET.

FIG. 11b

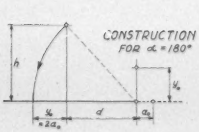


FIG. 11c

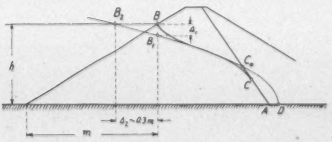


FIG. 11d

FIG. 11.— APPLICATION OF BASIC PARABOLA TO DETERMINATION OF DISCHARGE POINT OF LINE OF SEEPAGE.

e. Correction for Upstream Slope; Quantity of Seepage. Due to the entrance condition for the line of seepage and due to the fact that Dupuit's assumption is not valid for the upstream wedge of a dam, the line of seepage deviates from the parabolic shape. For the usual shape of a dam, there is an inflection point with a sharp curvature in the first section of the line of seepage, while for a vertical entrance face there is only an increase in curvature without reversal of direction.

For an accurate seepage analysis, these deviations should be taken into consideration. Referring to Figure 11d, it would be necessary to know in advance the position of one point of the parabolic curve in the vicinity of the entrance point B. L. Casagrande has chosen the intersection B_1 of the ordinate through the entrance point with the continuation of the parabolic line of seepage and has expressed the correction $\overline{BB_1} = \Delta_1$ as a function of d, h and the slope of the entrance face. A graphical presentation (11) facilitates the finding of the necessary correction Δ_1.

Somewhat simpler is the following approach. Instead of selecting point B_1 for the start of the theoretical line of seepage, we choose its intersection B_2 with the upstream water level. The corresponding correction Δ_2 is about $\frac{1}{3}$ to $\frac{1}{4}$ of the horizontal projection m of the upstream slope, or for average conditions $\Delta_2 = 0.3m$. This is easy to remember and dispenses with the necessity for tables or graphs for the correction. The determination of the line of seepage is then carried out with point B_2 as the starting point. The actual shape of the first portion of the line of seepage, starting at point B, can easily be sketched in, so that it approaches gradually the parabolic curve, as shown in Figure 11d.

The quantity of seepage q per unit of length can be computed either from equations (12) or (16). If we substitute in these equations the known quantities, they appear in the following form:

$$q = k\left(\sqrt{h^2+d^2} - \sqrt{d^2 - h^2\ cotan^2\alpha}\right) sin^2\alpha \qquad (17)$$

and

$$q = k\left(\sqrt{d^2+h^2} - d\right) \qquad (18)$$

For the great majority of cases encountered in earth dam design, both equations give practically the same result, so that the simpler equation (18) should be used for general purposes. In other words, the quantity of seepa is practically independent of the discharge slope, and is equal to the quantity that corresponds to the basic parabola. Only in those cases in which the starting point of the line of seepage is very near the discharge face will the difference between the two equations warrant the use of equation (17).

For the comparatively rare case in which the presence of tail water must be considered in the design, the determination of the line of seepage and of the quantity can be performed by dividing the dam horizontally at tail water level into an upper and lower section. The line of seepage is determined for the upper section in the same manner as if the dividing line were an impervious boundary. The seepage through the lower section is

determined by means of Darcy's law, using the ratio of the difference in head over the average length of path of percolation as the hydraulic gradient. The total quantity of seepage is the sum of the quantities flowing through the upper section and the lower section. The results obtained by this rather crude approximation agree remarkably well with the values obtained from an accurate graphical solution.

Those readers who are interested in data showing how the results of seepage tests agree with the computed line of seepage and seepage quantity, using the methods described in this chapter, should consult References 10 and 11.

G. Seepage through Anisotropic Soils.

By a combination of the various methods of approach that have been outlined, and with proper consideration of boundary conditions as summarized in Figure 6 and discussed more in detail in Appendix I, one can arrive at a reliable determination of the line of seepage through even the most complicated cross-sections of earth dams. The cross-section may consist of portions with widely different permeabilities; however, each homogeneous section in itself is assumed to be isotropic, that is, possessing the same permeability in all directions. Unfortunately, this is practically never the case. Even a uniform clean sand, consisting of grains of the usual irregular shape, when placed in a glass flume for the purpose of building up a model dam section, does not produce an isotropic mass. The grains orientate themselves in such a manner that the coefficient of permeability is not uniform in all directions but larger in a more or less horizontal direction. As a consequence, the entire flow net is markedly influenced, resulting in considerable deviations from the theoretical flow net for an isotropic material.

Only by using a very uniform sand consisting of spherical grains, and by making tests on a sufficiently large scale, to reduce the capillary disturbance, can one arrive at test results that are in good agreement with theory. For this reason most of the tests described in References 10 and 11 were carried out on Ottawa standard sand.

Soils, in their natural, undisturbed condition, are always anisotropic in regard to permeability even if they convey to the eye the impression of being entirely uniform in character. If signs of stratification are visible, then the permeability in the direction of stratification may easily be ten times greater than that normal to stratification. For distinctly stratified soils this ratio can be very much larger than ten.

When soils are artificially deposited, as in the construction of a dam or dike, stratification develops to a greater or less degree. Such stratification has always been recognized by engineers as being undesirable, and for this reason special construction methods have been developed to disturb or destroy it. The hydraulic-fill core, during its construction, is frequently stirred with long rods in order to break up stratification as much as possible.

Sheepsfoot rollers are effective in compacting earth fills without creating distinct stratification. However, in spite of such precautionary measures, a certain amount of stratification remains. In addition, it is practically impossible to eliminate considerable variations in the general character of the material in the borrow pit; especially variations in permeability, which will result in substantial variations in the permeability of the dam from layer to layer. These cannot be eliminated by thorough rolling. Even the most carefully constructed rolled earth dams possess a considerably greater average permeability in a horizontal than in a vertical direction. Therefore, thorough investigation of variations in the character of the borrow pit materials forms an important part of preliminary studies. Taking into consideration the uncertainties that are always encountered in dealing with soil deposits and cannot be completely eliminated by the most elaborate investigations, it is essential that we should be conservative in the assumptions on which the design of an earth dam is based. This requires special attention to the possible degree of anisotropy in the dam.

The question of seepage through anisotropic soils was investigated for the first time and solved by Samsioe (14) in 1930. Fortunately the solution is simple and lends itself readily to practical application. The flow net of an anisotropic soil does not possess the usual characteristics of a flow net. However, it can be reduced, by the application of an appropriate geometric transformation, to an ordinary flow net. Designating the maximum and minimum coefficient of permeability for an anisotropic soil as k_{max} and k_{min}, it can be shown mathematically (see Reference 16) that by transforming the entire cross-section in such a manner that all dimensions in the direction of k_{max} are reduced by the factor $\sqrt{\frac{k_{min}}{k_{max}}}$, or that all dimensions in the direction of k_{min} are increased by the factor $\sqrt{\frac{k_{max}}{k_{min}}}$, the problem is again reduced to a solution of Laplace's equation. In other words, the flow net in the transformed section has the same characteristic flow lines and equipotential lines as previously discussed in this paper. Among others, Forchheimer's graphical method and all approximate methods suggested in this paper are applicable to the transformed section. After having found the line of seepage, or the entire flow net, in the transformed section, it is a simple matter to project this characteristic flow net back into the true section, in which flow lines and equipotential lines will not generally intersect at right angles. It should be noted that the hydraulic gradient at any point of the flow net and the magnitude of seepage pressures can only be determined in the true section, while the distribution of pore pressures and of hydrostatic uplift can be derived from either section.

The quantity of seepage can be computed from the transformed section on the basis of the coefficient of permeability $\overline{k} = \sqrt{k_{min.}\, k_{max.}}$. For proof see Reference 16.

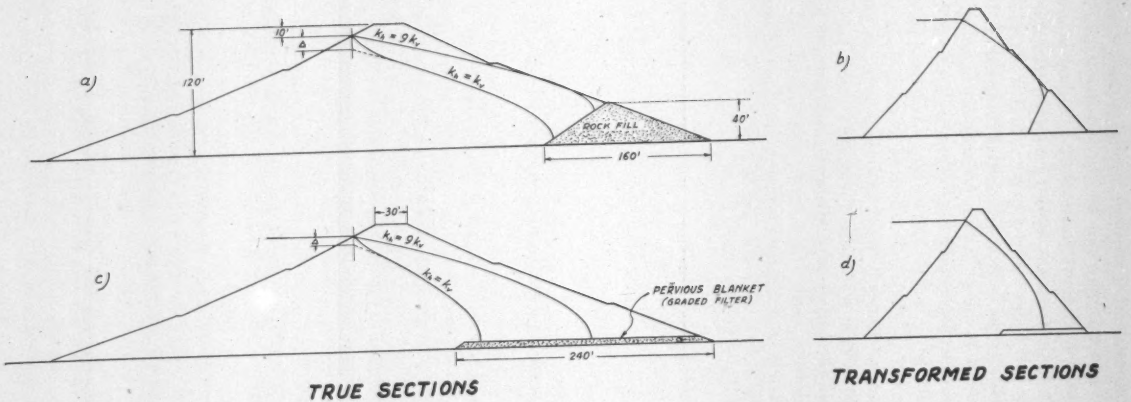


FIG. 12.— TRANSFORMATION METHOD FOR ANALYSIS OF STRATIFIED DAM SECTIONS.

Further information on seepage through stratified soils, the transformation theory, and examples may be found in References 15, 16 and 17.

The application of the transformation method is illustrated by the simple example of a rolled earth dam with a rock fill toe, shown in Figure 12. The dimensions and slopes of this dam are such that if a suitable soil is used, hardly any doubt would be raised regarding its stability. The rock fill toe seems to represent ample provision for safe discharge of seepage water. Indeed, the line of seepage, assuming isotropic soil, does fall well within the downstream face as shown in Figure 12a, (coefficient of permeability in horizontal direction k_h equal to coefficient in vertical direction k_v). However, if this dam is carelessly built of various types of soils with widely different permeabilities, a structure may well result that is many times more pervious in the horizontal direction than in the vertical direction. In the example shown in Figure 12, $k_h = 9k_v$ was chosen. On the right-hand side a new cross-section of the dam is plotted in which all horizontal dimensions are reduced by the factor $\sqrt{\frac{k_v}{k_h}} = \frac{1}{3}$. Then the line of seepage is determined in accordance with the methods outlined previously, and projected back into the true cross section. As can be seen in Figure 12a, the line of seepage for $k_h = 9k_v$ does intersect the downstream face, which is an undesirable condition that may in the course of time lead to a partial or complete failure of the structure.

H. Remarks on the Design of Earth Dams and Levees.

The question may arise of how to construct the downstream portion of a simple rolled earth dam, so that the line of seepage will remain a safe distance inside the downstream face of the structure, when only small quantities of coarse material are available. A simple solution is suggested in Figure 12c, in which a pervious blanket below the downstream portion of the dam is employed to control the position of the line of seepage to any desired extent. Such a blanket should be built up as a graded filter, carefully designed, to prevent erosion of any soil from the dam.

Whenever a dam or levee consists essentially of a uniform section of relatively impervious soil, e.g., possessing an average coefficient of permeability of less than 1×10^{-4} cm. per sec., the pervious blanket may well be extended as far as the centerline of the structure, as shown in Figure 13c. Such a design would add much more to the stability of the entire downstream portion, including the underlying foundation, than could be accomplished by a substantial flattening of the downstream slope. *A levee built in the conventional manner, with a downstream slope of 1 on 5, would possess less stability than a well-compacted levee in which the downstream slope is made as steep as 1 on 2, but which contains a filter blanket of the type shown in Figure 13c.* In the example illustrated in Figure 13c, it was assumed that a pervious foundation stratum lies beneath the levee and that the permea-

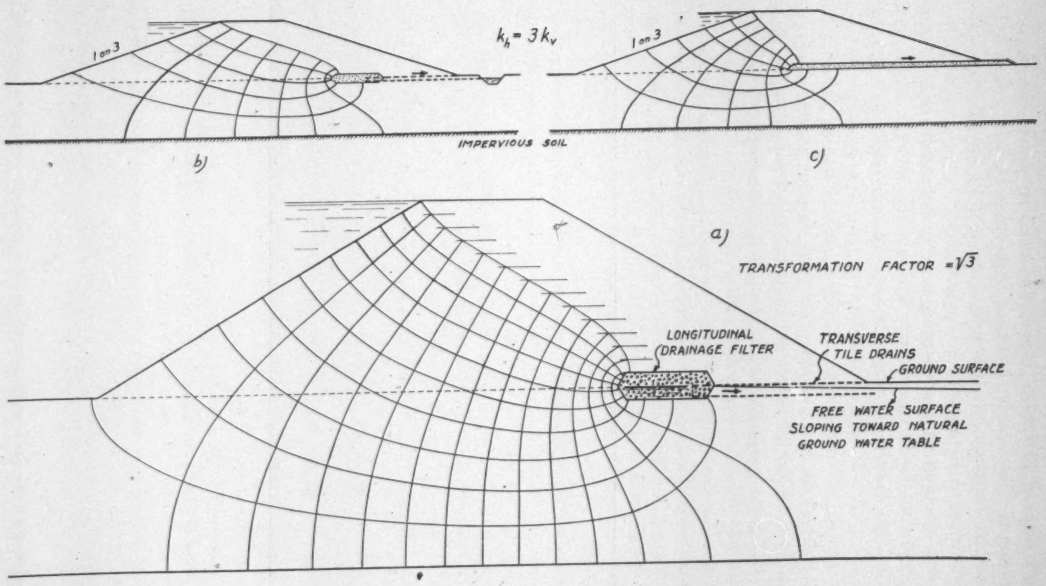


FIG. 13.— EFFECT OF INTERNAL DRAINAGE ON FLOW NET.

bilities of the levee material and the underlying foundation are the same, with a ratio of $k_h/k_v = 3$. The transformation which was required for obtaining the flow net, is illustrated by another example shown in Figures 13a and b. The assumptions are the same as in Figure 13c, except that the filter blanket is reduced to a longitudinal drainage strip with frequent transverse outlets. A tile drain may also be embedded within the core of the longitudinal drainage filter to increase the capacity of the drainage system if the structure consists of relatively pervious soils. This type of drainage would be employed where the quantities of suitable material for the drainage layer are very limited. To obtain the flow net, the true section was transformed into a steeper section, using as transformation factor $\sqrt{k_h/k_v} = \sqrt{3}$. The flow net was then obtained by Forchheimer's graphical method, by gradual approximation. Note the equi-distant horizontal lines intersecting the line of seepage. These were plotted before starting the flow net. In this example the line of seepage is not identical with the basic parabola, because the surface of the foundation on which the structure rests is not a flow line. However, it is convenient to use the basic parabola as a general guide for the first plot of the flow net. After a satisfactory solution is found, the flow net is projected back into the true cross section, Figure 13b.

No attempt is made in this paper to discuss in detail the important and interesting relationships that exist between the stability of earth dams or dikes and the seepage through and beneath them. However, it should be emphasized that the forces exerted by percolating water upon the soil can be very appreciable, and are often a maximum in critical points. These seepage forces are readily determined from a well-constructed flow net, and can then be combined with gravity forces for the stability analysis. Anyone who has made comparative studies of the seepage forces that may exist in dams and their foundations, must be impressed by the paramount importance of the design of those features that control seepage. It is not surprising that on the basis of empirical knowledge levees have been constructed with flat slopes. A levee built in the conventional manner of sandy soil, with slopes of 1 on 2 or 1 on 3, would be an unsafe structure. However, substantial flattening of the slopes is a very costly way of increasing its safety. Besides, even very flat slopes do not necessarily provide sufficient safety against undermining, particularly when a levee rests on a stratified, pervious foundation. In view of the large expenditures on levee construction which the next decade will bring, investment in research in this field would pay rich dividends if new designs for levees were developed that would not only be much safer than those built in the past, but considerably less expensive. *The widespread opinion among engineers that in earth dam and levee design "section makes for safety" needs to be revised.*

Many failures of levees are due to undermining caused by seepage through the foundation. Unless drainage provisions, as shown for example in Figure 13, are provided, the largest concentration of flow lines, both

through and beneath the dam, occurs at the downstream toe where the soil is not confined. As stated before, flattening the slopes does not greatly improve this condition. In some cases, particularly when the foundation soils are porous and distinctly stratified, drainage provisions within a levee may not be sufficient protection. In such circumstances it may be beneficial to drill frequent holes into the foundation beneath the future longitudinal drain, and to fill these holes with coarse material. Such "drainage wells" have been employed already for another purpose, the relief of upward pressure on an overflow dam (see Reference 18). Properly designed and constructed drainage wells would effectively destroy any serious flow concentration at the toe of the structure. In other cases, particularly for a very pervious, but relatively thin foundation stratum, the use of a sheet-pile cut-off may be the ideal solution.

Improvements in levee design, as suggested here, are of a nature that will not produce interference with modern construction methods. Drainage wells, longitudinal and transverse drains or drainage blankets must all be built before construction of a levee is started. The building of the levee can then proceed in the same manner as if the drainage structures did not exist, thus permitting full use of large drag lines and tower machines.

I. Seepage through Composite Sections.

For the purpose of controlling seepage and utilizing available soils to best advantage, it is usually necessary to build dams of several sections with widely different coefficients of permeability. Since it is common to require that the ratio in permeability between neighboring sections should be at least one to ten, it is rarely necessary to determine the flow net for the entire dam if a careful study is made of the least pervious sections. However, in some cases it may be necessary to determine the position of the entire line of seepage. In Figure 14 is reproduced an example given by L. Casagrande (12), showing the line of seepage for a combination of two sections, with the downstream section built of soil which is five times more

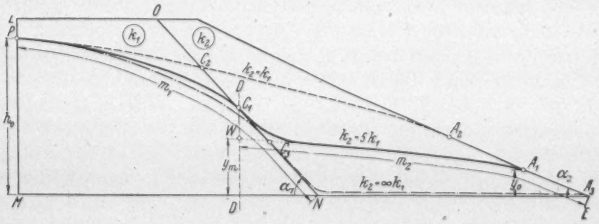


Fig. 14.— Line of Seepage for a Combined Section.
The computed result was verified by model experiments.
After L. Casagrande (11).

pervious than the central section. The line of seepage through such a composite section is found by changing the assumed position for the point of intersection of the line of seepage with the boundary until the quantities flowing through both sections are the same. After the correct position of the line of seepage has been determined, one can also develop any desired portions of the flow net.

J. Comparison between Forchheimer's Graphical Method, Hydraulic Model Tests and the Electric Analogy Method.

The purpose of model tests for seepage studies can be twofold (1) determination of the flow net for a given cross-section, assuming that the soil is isotropic; (2) determination of the flow net if the model is build up in such a manner that it resembles the prototype as to possible stratification, character of the soils, etc.

For the first purpose it is essential that the material used shall consist of grains as nearly spherical and as nearly of one size as possible. In addition, the models must be large enough so that the height of capillary rise will not distort the line of seepage, particularly in that portion of the flow net near the discharge point of the line of seepage. (See Figure 10.) Use of Ottawa Standard Sand has given good results. Since one should not go much below the size of Ottawa Standard Sand on account of the distorting effect of capillary rise, one is obliged to use artificial spheres, such as glass spheres, of appropriate sizes for models containing sections with different coefficients of permeability. For such coarse materials the validity of Darcy's law must be checked. The results of careful model tests conducted with such materials agree well with the solutions obtained by the graphical method, the electric analogy method, or rigorous theoretical solutions, so far as the latter are available.

For the second purpose, the testing of models similar to the prototype, one has to know first of all how the coefficient of permeability varies in the prototype, not only in its various sections, but particularly within each section due to anisotropy. Then one must build the model to imitate, on a small scale, these conditions. It is a waste of time and money to build a model using the same soil as in the prototype without attention to the anisotropic conditions in the prototype. Such a model does not represent the prototype; nor are the results comparable to the conditions for isotropic materials, because the inevitable irregularities and stratification due to the method of building the model are reflected in the resulting flow net to such an extent that the flow net looks very much like a beginner's attempt at employing the graphical method. The results of such model tests will lie somewhere between the conditions for an isotropic model and the actual conditions in the prototype and will tell practically nothing that can be of assistance in our problem; on the contrary, such

results are often very confusing, particularly when the tests are made by men inexperienced in theoretical and graphical solutions and, therefore, are unable to interpret properly the test results.

The electric analogy method, when used by an experienced operator, is a useful and accurate method for *direct determination of flow nets between given boundaries.* Composite sections consisting of soils with different permeability can also be investigated by this method. Unfortunately, the method does not permit the *direct* determination of the line of seepage for those problems in which the upper surface is not a fixed boundary. Another disadvantage of this method is that it requires an accurate apparatus and the construction of a special testing model for every problem. If compared with graphical solutions, the electrical method is more expensive and requires more time. It should also be mentioned that a satisfactory presentation of the results obtained by means of the electrical method requires knowledge and application of the graphical method. In some instances the amount of work required to transform the test results into a good-looking flow net would have been enough to produce this flow net by the graphical method without assistance of the electrical apparatus. The graphical method will serve as an excellent check on the electrical method and should be used whenever accurate solutions are sought.

Another important advantage of the graphical method is that the process of finding the flow net for a proposed section almost inevitably suggests changes in the design which would improve the stability of the structure, and often its economy. With some experience in the use of the graphical method the effects of changes in one or the other detail of the design can quickly be appraised without the necessity of finding the complete flow net for a number of different cross-sections. Thus there can be explored in a short time many possibilities which would require months of work with any of the other methods. Such studies have already indicated desirable changes from the conventional design of earth dams and levees, some of which were briefly discussed in the preceding chapter.

Finally, there should be mentioned the pedagogical value of the graphical method. It gradually develops a feeling or instinct for streamline flow which not only improves, in turn, the speed and accuracy with which flow nets can be determined, but also develops a much better understanding of the hydromechanics of seepage and ground water movement. The investigator who is trained only in the use of "mechanical" methods for analysing seepage problems can check his tests only by performing additional tests. He is rarely able to detect inaccuracies by the appearance of the test results. In contrast to this, the author has been able to point out even minor inaccuracies in the results obtained from model tests, as a result of the sense for streamline flow developed by applying the graphical method for years.

In concluding this discussion, the author wishes to emphasize the almost obvious point, which nevertheless is frequently overlooked, that the

investigator should consider carefully, before starting any model tests, what information he desires to obtain from these tests. In nine cases out of ten he will then come to the conclusion that he could obtain the results without tests. Particularly in those cases where he attempts to evaluate the effect of variations in the coefficient of permeability, he will arrive at a better conception of the probable limits within which the seepage conditions in the prototype may vary by making a careful study of the possible variations in the coefficient of permeability (e.g. from studies of the variations in the borrow pit material) and then applying these values in graphical solutions, utilizing the transformation method. A model test would yield only one result, the relation of which to the prototype is often unknown. Such a test would certainly not permit a conclusion in regard to the probable limits within which the actual flow conditions will vary.

The practical application of the graphical method would be promoted if, for all typical conditions encountered in dam design, carefully constructed flow nets were published. The beginner in the use of the graphical method in particular, would be greatly assisted and encouraged in his efforts to acquire skill in the use of this valuable tool.

Appendix I.

(*a*) *Deflection of Flow Lines Due to Change in Permeability.* Flow lines are deflected at the boundary between isotropic soils of different permeability in such a manner that the quantity Δq flowing between two neighbouring flow lines is the same on both sides of the boundary. Referring to Figure 5, in which the flow net is plotted on the basis of squares for the material on the left of the boundary, and designating by Δh the drop in head between any two neighbouring equipotential lines, the following relationship can be set up:

$$\Delta q = k_1 a \frac{\Delta h}{a} = k_2 c \frac{\Delta h}{b}$$

or

$$\frac{k_1}{k_2} = \frac{c}{b} \tag{19}$$

$$\frac{a}{\sin \alpha} = \frac{b}{\sin \beta} \text{ and } \frac{a}{\cos \alpha} = \frac{b}{\cos \beta}$$

By combining these relationships one arrives at:

$$\frac{c}{b} = \frac{\tan \beta}{\tan \alpha} = \frac{k_1}{k_2} \tag{20}$$

Expressed in words, the deflection of the flow lines occurs such that the tangent of the intersecting angles with the boundary is inversely proportional to the coefficients of permeability. Furthermore, the squares on one side of the boundary change on the other side into rectangles with the ratio of their sides equal to the ratio of the coefficients of permeability, such

that the flow channels are wider in the material with the smaller coefficient of permeability.

It is probable that Forchheimer was the first one to use these relationships. However, he never took the trouble to publish them. In 1917, he communicated those relationships to Terzaghi, who made extensive use of them in his foundation investigations of dams, and also taught them in his course in Soil Mechanics at the Massachusetts Institute of Technology, during 1925–29.

(*b*) *Transfer Conditions for Line of Seepage at Boundaries; General Remarks.* L. Casagrande (10) made use of the general properties of a flow

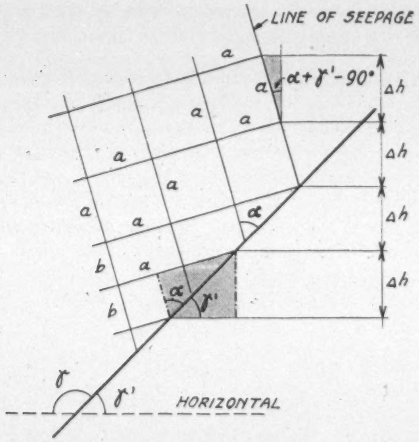


FIG. 15.— DERIVATION OF DISCHARGE CONDITION INTO OVERHANGING SLOPE.

net to analyze the condition at the entrance and discharge points of the line of seepage. Following the same general approach, the author determined the transfer conditions for other cases, including the transfer at the boundary between soils of different permeability. The results are assembled in Figure 6.

To acquaint the reader with the method used, it will be sufficient to present in the following the derivation for two typical cases.

(*c*) *Discharge into an Overhanging Slope.* In Figure 15 is shown the flow net in the immediate vicinity of the discharge point, sufficiently enlarged so that flow lines and equipotential lines appear straight. The slope of the line of seepage at the discharge point, the "discharge gradient,"

is assumed arbitrarily; then the flow net is plotted, starting with a series of equidistant horizontal lines which represent the head of consecutive equipotential lines. One can see immediately that the assumed discharge gradient in Figure 15 cannot be correct, because it is impossible to draw squares in the lower portion of the flow net. By setting up the condition that the sides a and b of the resulting rectangles must become equal, one can arrive at the necessary condition for the discharge gradient.

By projecting the sides a and b in the shaded triangles, Figure 15, one arrives at the following equation:

$$\frac{b}{\cos\alpha}\sin\gamma = a\cos(\alpha+\gamma-90°) = \Delta h$$

To fulfill the condition $a = b$, the only possible solution is $\alpha = 90-\gamma$; that means the line of seepage must have a vertical discharge slope.

(*d*) *Transfer Conditions for Line of Seepage at Boundary between Soils of Different Permeability.* To analyze the transfer conditions for the cases illustrated in Figures 6k and m, we start from the conditions that the hydraulic gradient at any point along the line of seepage is equal to the sine of the slope of the line of seepage of that point; and that the quantity flowing through a very thin flow channel along the line of seepage must be equal on both sides of the boundary. Referring to Figure 16a, we have the following velocities along the line of seepage, on both sides but in the immediate vicinity of the boundary:

$$v_1 = k_1 \sin(\alpha-\omega')$$
$$v_2 = k_2 \sin(\beta-\omega')$$

The quantity Δq flowing through the channel is:

$$\Delta q = a k_1 \sin(\alpha-\omega') = c k_2 \sin(\beta-\omega')$$

wherein the quantities a and c represent the widths of the flow channels, in accordance with Figure 16a. After replacing the quantities a and c by their projection onto the boundary, and substituting $k_1/k_2 = \tan\beta/\tan\alpha$, one arrives at the general condition:

$$\frac{\cos\alpha}{\cos\beta} = \frac{\sin(\beta-\omega')}{\sin(\alpha-\omega')}$$

or

$$\frac{\sin(90-\alpha)}{\sin(90-\beta)} = \frac{\sin(\beta-\omega')}{\sin(\alpha-\omega')}$$

If we assume $90-\alpha > \beta-\omega'$, then it follows that $90-\beta < \alpha-\omega'$, and vice versa. Hence the only possible solution is:

$$\left.\begin{aligned} &90-\alpha = \beta-\omega' \\ \text{or}\quad &\beta = 90+\omega'-\alpha \\ \text{or}\quad &\beta = 270°-\alpha-\omega \end{aligned}\right\} \qquad (21)$$

Since this condition does not contain the coefficients of permeability, it has to be fulfilled simultaneously with equation (20). Equations (20) and (21) determine, for a given slope ω of the boundary, the unknown angles α and β between the line of seepage and the boundary.

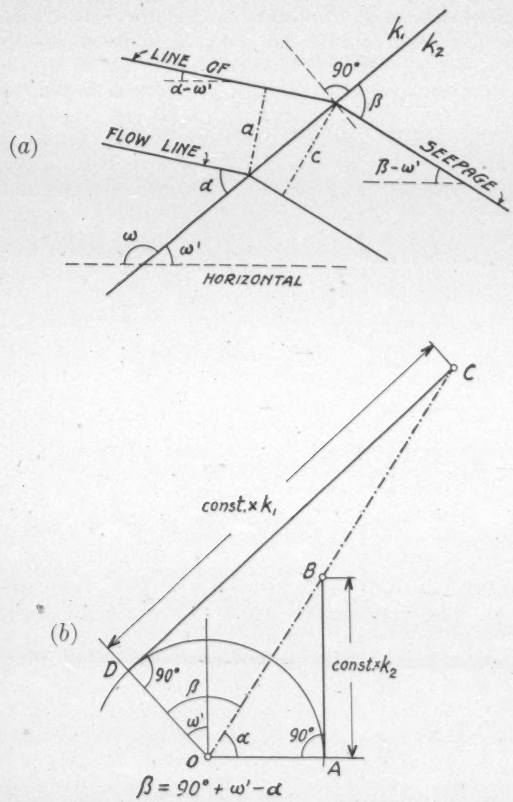


FIG. 16.— TRANSFER CONDITION OF LINE OF SEEPAGE AT OVERHANGING BOUNDARY.

The solution of these two equations can best be found graphically in the manner illustrated in Figure 16b. A circle is drawn with an arbitrary radius and the angle AOD is made equal to $(90+\omega')$. Then lines are drawn through points A and D perpendicular to the corresponding radii. The

problem is to draw another line through the center O (shown as dot-dash line) which fulfills the condition that the ratio $\overline{AB}/\overline{CD} = k_2/k_1$. Such a line can be found quickly by trial. The unknown angles α and β are determined by the angles between the dot-dash line and lines OA and OD, respectively.

Depending on whether k_1 is larger or smaller than k_2, we arrive at solutions in which either point B or point C is nearer the center of the circle. The corresponding deflection of the line of seepage is illustrated in Figures 6k and m.

While this theoretical solution for $k_1 > k_2$ can easily be verified by model experiments, it is not generally true for $k_1 < k_2$. In this case, when the downstream section is more pervious, the boundary condition for the line of seepage is also influenced by all other dimensions of the dam, especially by the elevation of the discharge point and its distance from the boundary under consideration. Only in special cases, particularly for high tail water

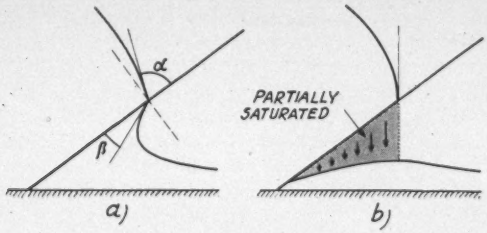


FIG. 17.— TRANSFER CONDITION OF LINE OF SEEPAGE AT OVERHANGING BOUNDARY.

level, and coefficients of permeability that do not differ greatly, does the line of seepage follow the theoretical solution. Whenever the theoretical solution has the appearance shown in Figure 17a, with the line of seepage deflected into an overhanging slope, it represents a condition that may be observed on a small scale in the laboratory but does not occur on a large scale. Instead of the continuous line of seepage of Figure 17a, a discontinuity develops, with the water seeping vertically into the more pervious soil, and only incompletely filling its voids. In other words, the quantity discharging vertically downward at the boundary is insufficient to fill the voids of the coarse material. Therefore, normal atmospheric pressure will act along that section of the boundary and the laws for open discharge are valid, forcing the line of seepage to assume a vertical discharge gradient at the boundary. That portion of the coarser soil which is only partially saturated, is illustrated in Figure 17b by the shaded area.

The graphical solution shown in Figure 16b also permits determination of the transfer conditions for the entrance of the line of seepage for the special case illustrated in Figure 6c. The open body of water on the up-

stream side may be considered a porous material with $k_1 \rightarrow \infty$, for which case Figure 16b yields $\beta = 90°$. The same conclusion may be reached from equation (21), remembering that for this case, $\alpha = \omega'$. This result means that the line of seepage enters perpendicularly to the upstream face of the dam, as shown in Figure 6c.

(*e*) *Singular Points in a Flow Net.* In trying to apply Forchheimer's graphical method, the beginner is frequently puzzled by the fact that some

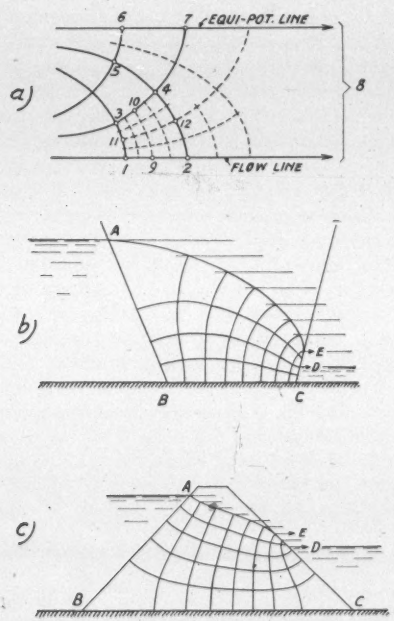


FIG. 18.— ILLUSTRATIONS OF SINGULAR POINTS OF FLOW NETS.

"squares" have no resemblance to real squares, and that, in some cases, flow lines and equipotential lines do not intersect at right angles. For example, in Figure 18a, the full-drawn areas 1, 2, 3, 4 and 4, 5, 6, 7 do not appear like "squares" to the inexperienced. However, by subdividing such areas by equal numbers of auxiliary flow lines and equipotential lines, one can easily check whether the original area is a "square" as defined for flow nets. By such sub-division one must arrive at areas which appear more and more like real squares. However, in most cases it is sufficient to compare

the average distances between opposite sides, that is, e.g., the lengths 9–10 and 11–12, by means of a pair of dividers.

In Figure 18a, the entire area to the right of points 2, 4, 7 must also be considered a square, in spite of the facts that the fourth point lies at infinity and that the angle between the flow line and equipotential line at this point is zero instead of 90°. It is, indeed, possible to continue subdividing this area, as shown by the dotted lines, always leaving a semi-infinite strip as the "last square." By this process of subdivision the amount of water entering into the "last square" is continuously reduced and approaches zero. In this way it is possible to reconcile the irregularity of the fourth corner by the fact that there is no flow of water at that point.

Similar irregularities in the shape of squares appear wherever a given boundary of the soil, with water entering or discharging, and boundary flow lines (impervious base or line of seepage) intersect at a predetermined angle. If this angle is less than 90°, then the velocity of the water at the point of intersection is zero. Such points are the entrance point A of the line of seepage in Figure 18b, and points B and C in Figure 18c. On the other hand, if the intersecting angle is greater than 90°, then the theoretical velocity in that point is infinite. Such points are corner A in Figure 9d, corners B and C in Figure 18b, and point D in Figure 18c. The last, representing the concentration of flow lines at the elevation of tail water level, is the cause for the well-known erosion which is observed on the downstream slope of homogeneous dam sections at the line of wetting.

At points where the theoretical velocity is infinite, the actual velocity is influenced by the facts that for larger velocities Darcy's law loses its validity, and that changes in velocity head become so important that they cannot be neglected. Hence, in the vicinity of such points the general differential equation (4) is not valid, and the flow net will deviate from the theoretical shape. However, the areas affected are so small that these deviations may be disregarded.

Appendix II.

Additions to the Original Paper.

(*a*) *Graphical Procedure for Determining Intersection between Discharge Slope and Basic Parabola.* The intersection between the discharge face and the basic parabola, designated in Figure 9 by point C_o, can be determined by the following simple graphical procedure.

The ordinate h_1 of the intersection of the basic parabola $x = \frac{y^2 - y_o^2}{2y_o}$ with the discharge slope $y = \pm x \tan \alpha$ is found as the solution of these two equations in the following form:

$$h_1 = \pm \frac{y_o}{\tan \alpha} + \sqrt{\frac{y_o^2}{\tan^2 \alpha} + y_o^2}$$

The first member, $\frac{y_o}{\tan \alpha}$, is equal to the distance $\overline{EB} = f$, in Figure 19; the second member, under the square root, is equal to the distance $\overline{AB} = g$; the ordinate h_1 of the intersection is simply equal to the sum $(f+g)$ for angles $\alpha < 90°$, and equal to the difference $(g-f)$ for angles $\alpha > 90°$. These relationships are expressed by the construction shown in Figure 19, which needs no further explanation.

The discharge point of the line of seepage is then found as discussed in Section F-d, with the help of Figure 11.

(*b*) *Comparison between Hamel's Theoretical Solution and the Proposed Approximate Methods.* Hamel (19), has succeeded in arriving at a rigorous mathematical solution of the problem of seepage through a homogeneous

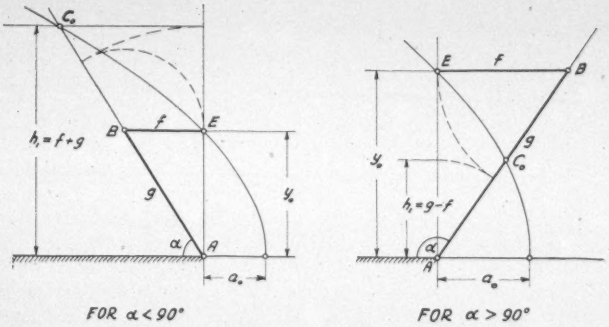


FIG. 19.— GRAPHICAL METHOD FOR DETERMINING INTERSECTION BETWEEN BASIC PARABOLA AND DISCHARGE FACE.

dam section. Unfortunately, the theory is so cumbersome that, in its present form, it is of little use to the engineering profession. It will be necessary to compute a sufficient number of typical cases, and publish the results in the form of tables or graphs, before engineers will be able to realize the advantages of this theoretical treatment. Recently, a few cases have been computed by Muskat (20) for coffer dam sections with vertical sides. These solutions presented an opportunity to investigate, at least for a few special cases, the accuracy of the approximate methods proposed in this paper. The results of this comparison were so encouraging that they are presented in the following paragraphs to permit the reader to formulate his own conclusions.

In Figure 20 are assembled three of the six cases which were published by Muskat (20). In each case the elevation of the discharge point, as computed from Hamel's theory, is designated by C_o, and its vertical distance

from the tail water level, or from the impervious base in the absence of tail water, is designated a_H.

An approximate elevation of the discharge point was found by means of the graphical procedure shown in Figure 7c. To facilitate the comparison between these figures, all points in Figure 20 are marked to correspond to those in Figure 7c. The construction is shown with full lines. The resultant discharge point is marked C, and its elevation from the base, or the tail water level, is marked a.

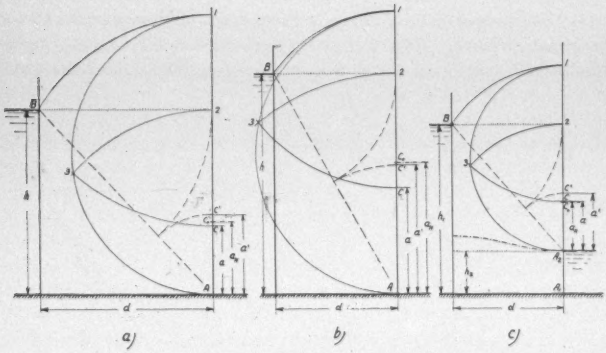


FIG. 20.—COMPARISON BETWEEN RIGOROUS AND APPROXIMATE DETERMINATIONS OF DISCHARGE POINT.

In addition to this construction, the simplified procedure was used in which $s_o = \sqrt{h^2+d^2}$. For a vertical discharge face the simplified formula for a becomes $a' = \sqrt{h^2+d^2} - d$ as proposed by Kozeny (6). The corresponding construction is shown in Figure 20 by dash lines and the resulting discharge point and elevation are designated by C' and a' respectively.

The case illustrated in Figure 20a, corresponding to Muskat's case No. 6, is identified by the ratio $d/h = 0.937$. Hamel's theory yields the quantities $a_H/h = 0.394$, and for the rate of seepage, $q_H = 0.539\ kh$.

As was shown by Muskat (20) and Dachler (23), the rate of seepage computed by means of Dupuit's formula $q = \dfrac{h_1^2 - h_2^2}{2d} k$, or without tail water $q = \dfrac{h^2}{2d} k$, represents an excellent approximation. For the case illustrated in Figure 20a, we have

$$q = \frac{h}{2d}\ kh = 0.539\ kh$$

The next case, Figure 20b, corresponds to Muskat's case No. 4 and is identified by the ratio $d/h = 0.556$. The theory by Hamel yields the quantities $a_H/h = 0.596$, and $q_H = 0.898\ kh$. The approximate rate of seepage, computed from Dupuit's formula, as described for the previous case, is $q = 0.900\ kh$.

The third case, shown in Figure 20c, is identical with Muskat's case No. 2. It differs from the other examples by the assumption of a definite tail water level, and is identified by the quantities $d/h_1 = 0.663$, and $d/h_2 = 2.81$. From the theory we get $a_H/h_1 = 0.301$, and $q_H = 0.717\ kh_1$. Dupuit's formula yields $q = 0.695\ kh_1$.

The comparison between the values for the elevation of the discharge point obtained by Hamel's rigorous solution and those by the approximate methods shows that, for engineering purposes, the approximate solutions are very satisfactory. It is interesting and of practical value to note that the approximate methods also give satisfactory results for ratios d/h considerably smaller than 1.0. Considering that the upstream portion of the flow net differs considerably from Dupuit's assumption of a constant hydraulic gradient in all verticals, this result is somewhat unexpected. For ratios of $d/h<1.0$, it appears that Kozeny's formula (13) gives slightly better results than the formula by L. Casagrande.

The remarkable agreement between the theoretical rate of seepage and Dupuit's approximate solution deserves special emphasis.

(c) *Graphical Solution by Means of the Hodograph.* A new graphical method for determining the flow net was proposed by Weinig and Shields (30), in which the flow-net is determined graphically in the "hodograph plane" and then projected into the actual cross section.

The hodograph of a flow line is the curve which one obtains when plotting from one origin velocity vectors for all the points of the flow line. Therefore, the straight line connecting the origin with one point on the hodograph represents the magnitude and direction of the velocity for the corresponding point on the flow line.

Since the velocity along the free water surface is proportional to the sine of the slope, the hodograph for the line of seepage is a circle with diameter equal to the coefficient of permeability. The hodograph for a straight boundary is a straight line. Therefore, all boundaries of the hodograph that correspond to the flow net of a homogeneous isotropic dam section are known, and it is possible to set up equations that represent the solution of the problem in implicit form. That such a theoretical solution is rather complicated, even for the simplest dam section, has been mentioned before in the discussion of Hamel's theory (19). Therefore, Weinig and Shields follow the theoretical approach, using the hodograph, as far as mathematics permits conveniently; then they proceed to find the flow lines and equipotential lines in the hodograph by a graphical procedure which is essentially similar to Forchheimer's method.

One advantage of the method by Weinig and Shields is the possibility of determining numerically correct values for the velocity at certain points along the boundaries. In comparison with Forchheimer's graphical method the approach by Weinig and Shields is much more complicated and requires a thorough acquaintance with the hodograph, which very few engineers possess. Furthermore, this method is limited to simple cross-sections, while Forchheimer's method can be applied to complicated dam sections and foundation conditions.

Weinig and Shields (30) have solved a steep triangular dam section by means of the graphical solution of the hodograph. This cross-section is

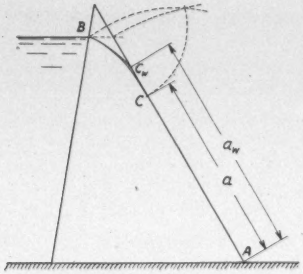


FIG. 21.— DISCHARGE POINTS OBTAINED BY GRAPHICAL SOLUTION OF HODOGRAPH AND METHOD ILLUSTRATED IN FIG. 7.

illustrated in Figure 21. Point C_w represents the discharge point as determined from the hodograph, and point C the discharge point using the method shown in Figure 7b. The elevation of point C is 15 per cent. lower than that of C_w. How much of this difference is due to inaccuracy in one or the other method is uncertain. Probably the hodograph solution is more accurate when the entrance point of the line of seepage is very close to the discharge face. In Figure 20, as well as in Figure 21, the discharge points obtained by the simple graphical procedure are situated lower than the other more accurate solutions. This would indicate the necessity for applying a correction in those cases where upstream and downstream face are very close.

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CONSTRUCTION OF A MODIFIED SLOW SAND-FILTRATION PLANT AT GREENFIELD, MASS.

BY H. L. FIELD.*

[*Read September 24, 1936.*]

The earlier water works at Greenfield, Mass., and additions prior to 1915 have been described by George F. Merrill,† in a paper before this ASSOCIATION. The main supply now, as then, is taken by gravity from the catchment area of Glen Brook, a tributary of the Green River in the town of Leyden. This is supplemented by a ground water supply in the valley of the Green River, from which water is pumped. A 2½-m.g. concrete covered reservoir, about 200 ft. above the center of town and close to it, equalizes pressures and gives excellent fire service.

In 1915, the town was supplied with sufficient water and had greatly improved fire service, due to additions and improvements. Nevertheless the water from the upland reservoirs was extremely turbid at times, necessitating pumping from Green River during these periods as well as during dry periods when the storage in the reservoirs became depleted. In many years the entire pumping costs were necesitated by turbid conditions in the upland reservoirs.

The general arrangement of the works is shown in Figure 1. There is only a small amount of ground water storage in the catchment areas of Glen Brook or Green River, since these areas are mostly in slate rock formation. It is possible that the low flow of Green River may reach 1 m.g.d. These conditions and the scarcity of good storage sites increase the difficulty of supply.

In 1927, Allen Hazen reported on additional supply. His conclusions were based on a population of 50 000 people. The first step was to be the development of the Glen Brook supply by the construction of a 400-m.g. reservoir with its flow line at elevation 715, 189 ft. above that of the present upper reservoir. This was to be followed by the development of Green River.

The high cost of the works recommended—$28 to $39 per capita, based on a population of 50 000—deferred action until such time as operating costs of the present ground water supply, with possible supplements warrant the larger expenditure.

WATER PURIFICATION PLANT.

A study of a water purification plant for treating the waters of Glen Brook was begun in January, 1932.

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*Engineer and Superintendent, Greenfield Water Department.

†Then Superintendent, Greenfield Water Department. "The Greenfield Water Works." This JOURNAL *29*, 149 (1915).

Population and Consumption. The report of Hazen, presented in 1927, did not reckon with the depression and the installation of meters on services. Therefore estimates of population and consumption are interesting in view of present conditions, as shown in Table 1.

Character of Water. The quality of the water from the upper reservoir, from which the supply for the water purification plant is taken has been

TABLE 1.— POPULATION AND CONSUMPTION 1900 TO 1950.

Year.	Population.		Water Consumption. Million Gallons Daily.	
	Hazen's Estimate.	Present Estimate.	Hazen's Estimate.	Present Estimate.
1900	7 927	7 927	....	....
1910	10 427	10 427	....	....
1920	15 462	15 462	1.62	1.62
1930	20 000°	15 550	1.73°	1.41°°
1940	25 500°	20 200°	2.31°	2.11°
1950	30 300°	26 300°	3.10°	2.61°
1960	42 600°	34 500°	4.12°	3.25°

°Estimated. °°Meters installed 1928–1929.

TABLE 2.— QUALITY OF WATER IN UPPER LEYDEN RESERVOIR.

Determination.	Parts per Million.		
	Maximum.	Average.	Minimum.
Turbidity	v.slight	v.slight	0
Color	15	8	0
Nitrogen as free ammonia	0.072	0.038	0.008
Nitrogen as albuminoid ammonia—total	0.098	0.080	0.054
Nitrogen as albuminoid ammonia—suspended	0.034	0.021	0.010
Chlorides	3.8	2.2	1.3
Hardness	50	41	33
Iron	0.18	0.10	0.05
Total residue on evaporation	76.0	69.5	63.0
Mineral residue on evaporation	63.5	52.7	37.5
Volatile residue on evaporation	12.5	16.8	25.5

analyzed by the State Department of Health, results being summarized in Table 2.

At times of storm, high turbidities occur. Samples collected from the tap in the Water Department office at times had a turbidity of 20 or more, while the reservoir water at times had a turbidity of 150 or more, for short periods.

In Hazen's opinion, the water required chemical treatment and filtration, using rapid filters. This was the proper choice, leaving attendance out of consideration, but with a plant necessarily located on a hillside nearly five miles from town, and with no provision for constant attendance, it was thought best to consider the use of slow filters, with chemical treatment whenever the character of the water should require it. Ordinarily the water is fairly clear, and has a color less than 10 p.p.m., a condition easily handled by slow filters.

Thinking there was ample time for choice of types, it was planned to operate an experimental plant to determine whether or not the higher first cost of the slow filters was justified and to demonstrate whether the turbidity of the Glen Brook water was a serious factor as affecting design. Slow filters have the advantages of not requiring the use of chlorine or attention at frequent intervals, and of producing with proper pre-treatment, a safe and agreeable water at all times.

Funds were not available for the experiments, and the selection of type of filter was held in abeyance until unemployment suddenly brought the Department to constructing the works under the F. E. R. A. Instead of basing the choice of type upon experimental data, it was based on general knowledge and experience. Instead of a deliberate design, an outline plan was hastily sketched and the plant located in a few hours, a topographical map of the site fortunately having been prepared previously. The work was begun, and the plans prepared afterwards.

The location of the plant with reference to the rest of the water works is shown in Figure 1.

Design. The plant is designed for a capacity of 2 m.g.d. and a filter rate of 5 m.g.d. per acre. It consists of a coagulating basin in two units, each storing 9.6 hours of flow, two filter units, each 0.2 acre in area, a filtered water basin storing about 550 000 gal., a filter house, piping and appurtenances. (See Figures 2 and 3.)

Influent Main. A 20-in. main connecting with the supply mains from the upper reservoir enters the filter house where, on occasions of high turbidity, sulphate of alumina is added. The water then passes out of the filter house through the 16-in. influent main to the two coagulating basins, entering through an inlet trough along the wall of each basin. This trough is 2 ft. by 18 in. in area, and its bottom is perforated with 2-in. holes for better mixing and distribution of flow. Each basin is 80 by 48 ft. in area and has a working depth of 14 ft. The bottom is sloped to drainage outlets provided with 12-in. plug valves and discharging into a system of drains beneath the floor of the basin.

At the outlet of each basin is a trough 2 by 3 ft., perforated with holes like the inlet trough. There is an overflow pipe with its top at an elevation 0.25 ft. above the flow line of the basin. The elevation of water in the basin is controlled by a hydraulic valve in the filter house. The water from the

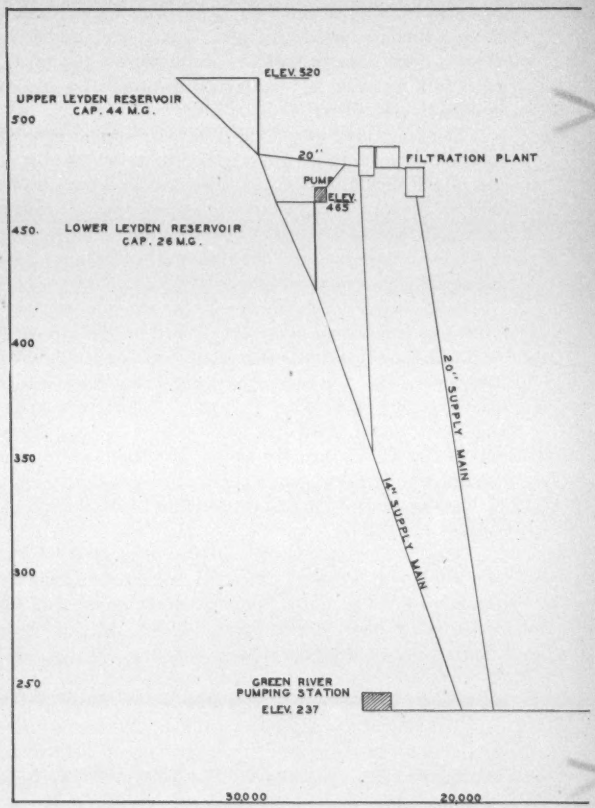
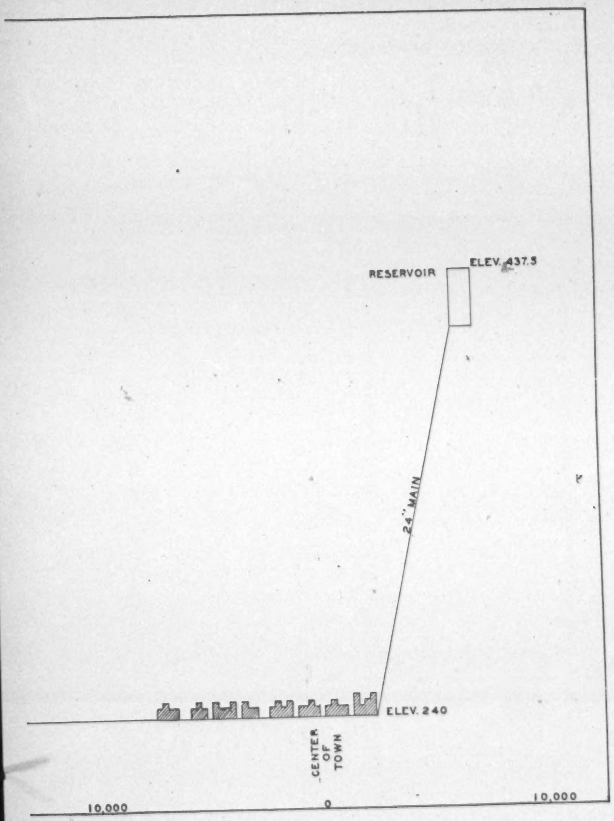


FIG. 1. — WATER SUPPLY



System — Profile.

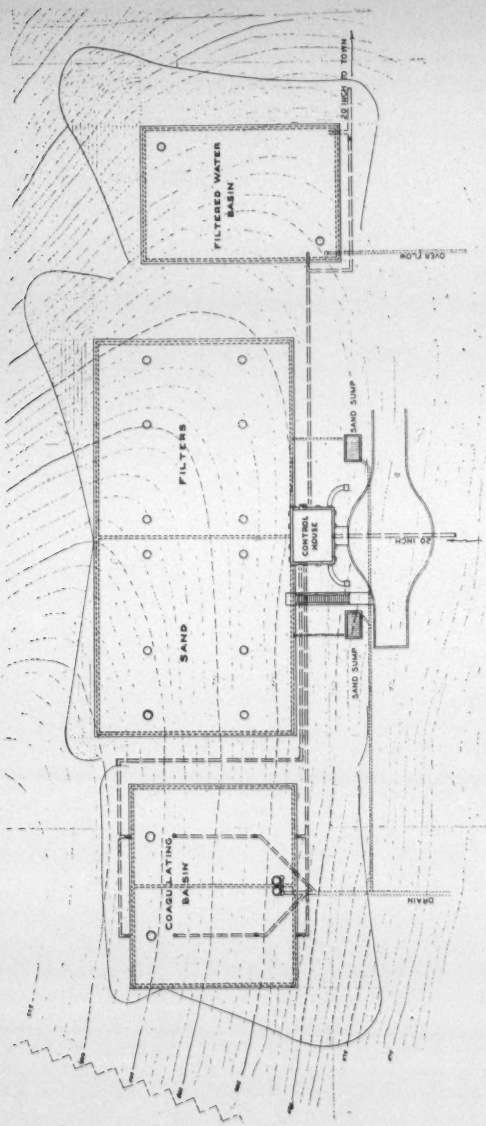


FIG. 2.—WATER FILTRATION PLANT—GENERAL PLAN.

coagulating basin re-enters the filter house and discharges through two 12-in. pipes with hydraulic valves into the two filter influent wells.

The basins are built of reinforced concrete, with the walls 15 in. thick, and covered with a flat slab roof 7.5 in. thick, supported on reinforced concrete columns 16 in. square, having expanded capitals at their tops, 6 ft. square. Manholes 3 ft. in diameter are provided for access, and the roof is covered with 18 in. of earth.

Filters. Each of two filters, which adjoin the coagulating basin, is 96 ft. square, which, with the area of inlet wells and columns deducted, is equivalent to about 0.2 acre, or 0.4 acre in both.

The design of roof, wall and supporting columns is similar to that of the coagulating basin. The columns are spaced 18 ft. center to center.

The floor is in the form of inverted groined arches, with six 8-in. lateral underdrain channels in each unit discharging into a 12-in. transverse main channel leading to the effluent main in the filter house. These channels are of varying depth to produce a uniform rate of filtration through the filter sand above; they are covered respectively with 8- and 12-in. split vitrified tiles.

Above the drains are three layers of graded gravel, as follows:

Depth — in.	Sizes of Stones — in.
7	1½ to ¾
3	¾ to ¼
2	¼ to #12 mesh

Above the gravel is a sand layer 3 ft. thick, the sand having an average effective size of 0.36 mm. and a uniformity coefficient of 2.5. The depth of water above the sand is 4 ft.

The water enters the filter through an influent well in the form of a 4-ft. quadrant placed at the corner of each filter. The wall of this well is of concrete, topped by three courses of brick, which may be removed, or replaced, course by course, as the elevation of the filter surface is lowered by scraping or raised by restoring the sand. Initially the top course was 0.21 ft. above the sand surface.

Sand Handling. In the front wall of each filter is a sand hopper for the reception of scrapings. These are washed by a jet from the hopper through a 6-in. drain to a sand sump in front of each filter, from which the sand is hauled away. Each sump is 12 by 8 ft. in area and 6.5 ft. deep. Stop planks are provided for the retention of sand, while the water escapes through a 10-in. tile drain. Removable plank covers furnish access to the sump.

Water from the filters passes through Builders Iron Foundry rate-of-flow controllers to the 16-in. main leading either to the filtered water basin or to the supply main to the town. The filtered water basin, 96 x 64 x 12 ft., with a capacity of about 550 000 gal., is of the same general construction as the coagulating basin, but with a flat floor instead of a sloping one. It is located to the south of the filters. Filtered water enters the basin through

the 16-in. pipe and flows out into the 20-in. main supplying the town. A 16-in. by-pass connects the influent and effluent pipes of the basin; a 12-in. overflow and 6-in. drain are also provided.

Filter House. The basement of the filter house is of concrete, the superstructure of brick, with brick pilasters and brick and cast stone trimming. (See Figure 3.) In the well-lighted basement are all the filter connections,

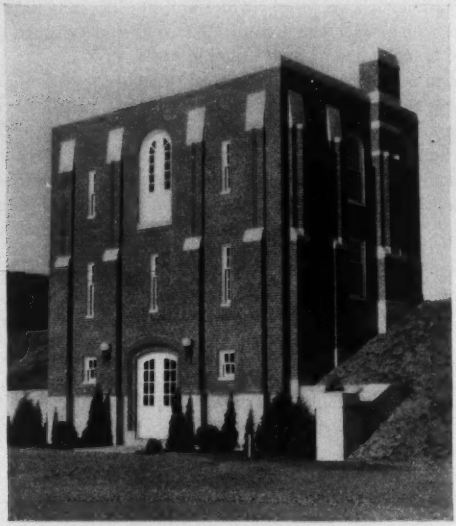


FIG. 3. — FILTER HOUSE AT NEW FILTRATION PROJECT.

valves and regulators, also the small pump and tank for supplying water for operating the hydraulic valves, dissolving chemicals and general usage. In the basement there is also an oil heater, which is connected with Modine heater units in various parts of the house.

In the control room above the basement are located the operating tables with valve levers, rate-of-flow and loss-of-head gauges, the chemical feeder, the indicator and recorder for the Venturi meter, and the apparatus and equipment required for observing, testing and recording the operation of the plant. The feeder is a Wallace and Tiernan M.O.F. dry feeder and is charged from the floor above through a chute and discharges through a dissolving box supplied with water, and connected with a pipe which discharges in turn into the influent pipe leading to the coagulating basin. The indicating and recording gauges for the Venturi meter are conveniently located on the wall of the control room.

Above the control room, storage for chemicals is provided. Chemicals are lifted from a truck by an electric hoist and are weighed and then discharged through the hopper supplying the chemical feeder on the floor below.

Elevations. The principal elevations are as follows:

Point.	Elevation — ft.
Upper Reservoir, crest of dam	526
Lower Reservoir, crest of dam	465
Proposed Reservoir Glen Brook, crest of dam	715
Crest of dam, Green River	242
Pumping Station Floor	237
Rocky Mountain Reservoir, flow line	437
Rocky Mountain Reservoir, bottom	413
Coagulating basin, flow line	490.5
Filters, flow line	490.0
Filtered Water Basin, flow line	480.0
Filtered Water Basin, bottom	468.0
Top of filter embankment	495.0
Roof of filter house	513.2
Basement floor of filter house	478.0

Because the lower reservoir is below the flow line of the water purification plant, a pump has been provided to raise the water from the reservoir to the hydraulic grade of the raw water main, thus making available the storage in the lower reservoir.

Operation. The plant was placed in service on November 1, 1935, and has been in continuous operation since. For most of the time no coagulant has been added to the water, but high turbidities have required the following applications of alum:

Date.	Turbidity. Raw Water — p.p.m.	Sulphate of Alumina. Lb. per m.g.	p.p.m.
1/ 3/36 to 1/ 5/36	35	100	12.0
3/12/36 to 3/13/36	60	185	22.2

The contract for electric power which includes a stand-by charge, allows running the pump at the Green River Station for about 48 hours a month, without extra cost. Consequently it is possible without increased cost of operation to operate this pump at the peaks of turbidity, thus avoiding treating the water at its worst, and saving chemicals.

The results of operation from November 1, 1935 to May 25, 1936 are shown in Table 3:

TABLE 3.— RESULTS OF OPERATION.

Days in Service	208	
Average rate of filtration — m.g.d.		1.56
Number of scrapings, both units		2
Number of rakings, both units		0
Yield, million gallons between scrapings:	136 days,	212.7
	*72 days,	112.6

*Period of 72 days includes flood period.

The character of the raw and filtered water on April 27, 1936 is shown in Table 4.

Construction. On November 20, 1934, the matter of the construction of a water filtration plant was presented to the Board of Water Commissioners, and by December 15, approval had been secured from the F.E.R.A. and funds voted by the town. Actual construction work was

TABLE 4 — CHARACTER OF RAW AND FILTERED WATER.

Constituent.	Parts per Million. Raw Water.	Filter Effluent.
Turbidity	very slight	0
Color	3	0
Sediment	.....	0
Ammonia Nitrogen:		
Free	0.004	0
Albuminoid	0.046	0.010
Chlorides	1.6	1.6
Nitrogen:		
As Nitrates	0.10	0.08
As Nitrites	.....	0.000
Hardness	33	33
Iron	0.10	0.05

started January 2, 1935, and the plant was completed and in operation November 1, 1935, nine months after starting work.

All pipe work and installation of equipment was done by Water Department employees and all key men directing important work were employed directly by the department. F.E.R.A. foremen were responsible to someone employed by the department. It was soon found that this was the only way in which this type of work could be accomplished successfully under the conditions by which labor was furnished. For a large part of the time during construction 250 men were employed. The labor furnished was permitted to work only 24 hours per week. It was found that it was more advantageous to split this number of men into two shifts of about 125 men each. In this way work could proceed continuously for six 8-hr. days each week. In order that a new crew of men should not start work every three days, arrangements were made so that men furnished by the F.E.R.A. could work their two three-day weeks without interruption and then not return to work until the other shift had worked their two three-day weeks. This worked out quite satisfactorily. The men employed by the department worked with both shifts for a full 48-hr. week.

Cost. The total cost of the plant and the installation of 1¼ miles of 20-in. cast-iron main was $176 710.41; of this amount $89 483.57 was furnished for labor by the F.E.R.A. and $87 226.84 was spent by the Water Department for materials, labor, land, engineering and miscellaneous items. The total cost of the 20-in. pipe line was approximately $40 000 and the total cost of the filtration plant alone was approximately $136 000.

The work was done under the direction of the writer. The plant was designed by Weston & Sampson, Consulting Engineers of Boston; Mr. Ralph C. Henry was consulting architect for the filter house.

During ten months of operation, the plant has produced water of excellent quality and practically zero in color.

The cost of operating the plant will not exceed $1 000 a year, and this is offset by savings in operation of the pumping station on Green River, to avoid turbid water from the reservoirs. In less than one year these savings have already amounted to more than $3,000.

DISCUSSION.

ROBERT SPURR WESTON.* After reading the author's paper, the first question one asks is, "Why slow filters?" This question comes naturally, in view of several statements by competent engineers that slow filters are out of date, particularly where chemical treatment is required and construction costs are from 75 to 100 per cent. higher than for rapid filters.

This question, regarding Greenfield, has been answered by Mr. Field in the following way:

(1) They avoid the necessity for constant attendance.

(2) The color of the water is low and chemical treatment to control the turbidity is required for but part (so far, but 1 per cent.) of the time.

(3) Treatment of the filtered water with chlorine is not required by the State Department of Public Health.

(4) Misfits in treatment are less serious in their effects.

(5) The difficult disposal of waste wash-water from a plant located away from a water course is avoided.

Like conditions were encountered at Northborough, Mass., where a similar plant was placed in service on September 2, 1936. In this latter case, however, a polluted, highly-colored water from an upland reservoir is treated with chemicals continuously, but, as at Greenfield, constant attendance could not be well provided. Both plants are in locations of difficult accessibility, especially in bad weather. Consequently, the postponement of filter scraping may be made without the serious consequences that would follow failure to wash rapid filters at the proper times.

As everyone knows, construction under the auspices of the F.E.R.A. has not always been attended with the good results obtained at Greenfield. Frequently, work under these auspices has suffered because of incompetent management or unskilled or unwilling labor. At Greenfield, however, many visitors have praised highly the character of the work and the results accomplished.

For the successful management the author of the paper is mainly responsible, not only for his selection of proper keymen as leaders, but for his foresight and judgment throughout the whole period of construction.

*Consulting Engineer, Boston, Mass.

He was fortunate in having among the laborers many skilled mechanics from local tool works which were idle. The precision with which these men were accustomed to work was reflected in the new work. This was especially noticeable in the finished concrete surfaces and in the trimness of the minor fittings of the plant. But, while ably assisted, it was the author's leadership and assumption of responsibliity that inspired the results and saved the money.

OPERATING EXPERIENCES AT THE SALEM AND BEVERLY FILTRATION PLANT.

BY ROBERT P. JOHNSON.*

[*Read February 18, 1937.*]

The Salem and Beverly Filtration Plant was placed in service on October 27, 1935. Two days later, a break in a special casting on the raw-water pipe-line temporarily placed the plant out of commission until November 1. Since then, however, the plant has been in continuous operation.

The physical aspects of the plant have been described in a paper presented before this ASSOCIATION by Marston and Raymond.† Except for a necessary brief description of the plant, therefore, this paper will be confined to a review of the operating experiences encountered during the past year.

WATER SOURCES.

The direct source of water supply is Wenham Lake, on the southerly shore of which is located the intake tower. As this supply alone is inadequate, it is replenished by takings from Longham Reservoir and the Ipswich River.

Longham Reservoir. Longham Reservoir, built in 1895, is located in Longham meadow. Here a 14-ft. earth dam was built across the Miles River. It has a capacity of 55 m.g., a drainage area of 3.28 sq. mi., and a surface area of 43 acres. Its water flows by gravity through a 36-in. cast-iron pipe into Wenham Lake.

Ipswich River. The drainage area of the Ipswich River at the point of taking is approximately 99 sq. mi. The diversion of water from the river is accomplished by means of a 10 000-ft. canal extending to the base of the ridge separating this watershed from that of Wenham Lake. At the end of the canal is located a pumping station containing two pumps, one of 6-m.g.d. and one of 25-m.g.d. capacity. From here the water is pumped through a 2 500-ft. varying size conduit over the ridge to the outlet on the northerly end of Wenham Lake.

Under the enabling statute of the Salem and Beverly Water Supply Board, water that is in excess of 20 m.g.d. may be diverted during the months of December through May. Later legislation now allows the diversion of water during the other six months when in the opinion of the State Department of Public Health this is necessary. The total amount taken in any one year, however, must not exceed 2 500 m.g.

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†THIS JOURNAL, 50, 111, (1936).

Wenham Lake. Wenham Lake is a natural reservoir of 800-m.g. available capacity. It has a drainage area of 3.5 sq. mi. and a surface area of 250 acres at full lake level. No stream of consequence enters it; it is fed mainly by springs.

Quality of Raw Water.

Longham Reservoir. Longham Reservoir is very shallow and, when full, covers large areas of peaty meadow land. During the summer months it is hardly more than a brook; at which time luxuriant growths appear on the sloping shores. As a consequence, when the reservoir refills in the fall, the water is highly colored and possesses an objectionable swampy vegetable odor.

Microscopic organisms thrive in the warm shallow water, especially diatoms and blue-green algae. Their presence, however, is not important, due to the insufficient reservoir capacity for water takings during the summer months.

Although there is no direct pollution of the reservoir, the headwaters flow through inhabited farm country, with the consequent opportunity afforded for contamination. It is not uncommon, therefore, to find members of the coli-aërogenes group present in 1-c.c. sample portions.

Ipswich River. The watershed of the Ipswich River is highly populated and includes many acres of swamp and marshland along the course of the river.

The daily flow of the river varies over wide limits. Both the maximum and minimum record flows were broken during the year 1936. During the March floods, rates of flow well in excess of 900 m.g.d. were recorded, whereas during August, flows under 4 m.g.d. were registered. With such wide variations in flow, therefore, the water is of a distinctly inferior type, possessing high color, objectionable odors, and containing large numbers of bacteria, including many of the coli-aërogenes group.

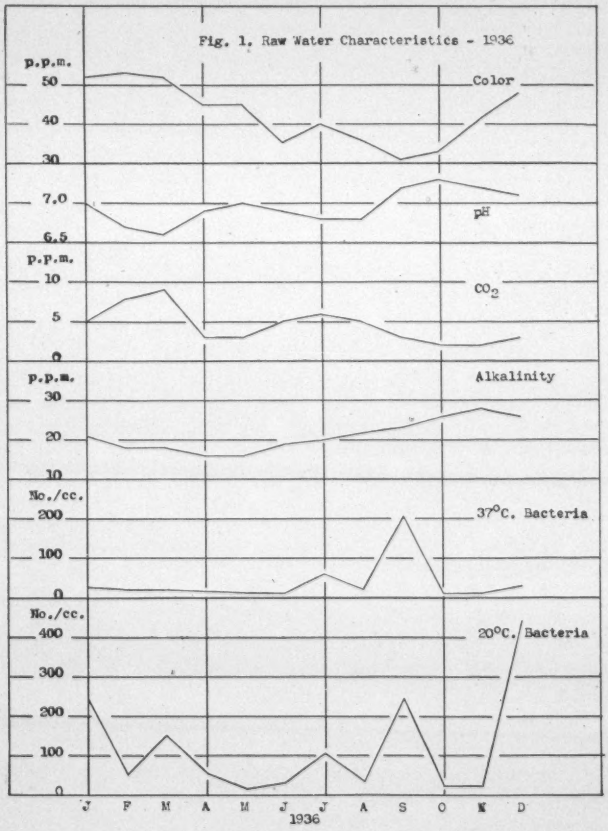
Wenham Lake. Early records of Wenham Lake show the water to have been of exceptionally good quality. The color was usually less than 10 p.p.m., odors were infrequent, and bacteria were few. The effects of the introduction of the inferior Longham Reservoir water after 1895 were soon noticed. The average color increased yearly, and odors due to organic matter and microscopic organisms became pronounced. This condition grew worse much more rapidly after accessions from the Ipswich River were started in 1913.

Prior to the construction of the filter plant, the average color had increased to 40 p.p.m., four times what it was before 1895. Odor troubles had become general. These had three origins. The growth of microörganisms, especially *Anabaena* and *Asterionella*, became yearly more severe. Continued takings from Longham Reservoir and the Ipswich River increased the organic odors peculiar to these waters. Thirdly, the inability to obtain water from the tributary sources during years of low rainfall often

caused the lake to drop to extremely low levels. This laid bare many acres of lake bottom from which objectionable odors arose.

The water had also become more corrosive. This may be attributed to the introduction of the highly colored tributary waters. Whereas the bottom of the lake was formerly clean gravel, it is now covered by precipitated organic matter. The putrefaction of this increases the carbon dioxide content of the water.

The numbers of bacteria increased, including members of the coli-aërogenes group. This is probably due to increased travel over the watershed and the introduction of water from the other sources, despite the chlorination of these sources. The chemical and bacterial characteristics of the lake water during 1936 are as shown in Figure 1.



Purification Processes at the Present Time.

The Salem and Beverly Filtration Plant is of the rapid sand-filter type. It is designed for 8 m.g.d. capacity at a 125-m.g.a.d. rate, or 9.6 m.g.d. capacity at a 150-m.g.a.d. rate. Provisions have been made for doubling the capacity by future extensions.

Intake. The intake formerly used by the Salem pumping station has been taken over by the Board. In the structure are two intake ports whose center lines are 10 ft. and 25 ft. respectively below high-water lake level. A gated connection into the former suction line has been made. From this point a new 36-in. pipe line extends to the low-lift pumping station.

Low Lift Pumping Station. The low lift pumping station, near the lake shore, is a small one-story brick building of colonial design. Here are located the three low-lift pumps, two vacuum pumps, and the main electrical switchboard. The three low-lift pumps are remote-controlled from the main filter building.

The pumps discharge into a 24-in. cast-iron pipe line, 900 ft. long, extending to the filter plant. The water is measured by a Venturi meter, and the flow is regulated by a float-controlled butterfly valve.

Mixing Tanks. There are two mixing tanks of concrete 18 ft. square by 17.5 ft. deep. Each is provided with a slow-motion motor-driven variable-speed stirrer, revolving about a vertical axis. The paddles are of wood, set at an angle of 60 degrees to the horizontal. They rotate counter to the flow of water through the tank. The tanks may be operated in series, in parallel, or singly. When both are operated, the total detention period is 15 minutes at an 8-m.g.d. rate.

Coagulation Basins. There are four concrete covered settling basins operated in parallel, each of which is 95 ft. long, 35 ft. wide, inside, and of 15-ft. average water depth. A longitudinal wooden baffle divides each basin, thus producing a distance of flow equal to twice the length of the tank. The basins may be cleaned either by completely draining the contents to the lagoons, or by pumping the supernatant water of one basin to the other basins, and draining off the sludge to the lagoons.

Wash Water Lagoons. There are three open lagoons located in part of a swamp adjoining the lake. They were constructed by excavating the peat and muck therein contained, and building earth embankments around the area. These lagoons operate in series, from which the clarified water of the third lagoon flows into the lake. Because of seepage and evaporation, however, this overflow to the lake has not yet taken place.

Filters. There are eight concrete filter boxes, each of which has a filtering area of 366 sq. ft. At the usual rate of 125 m.g.a.d., the capacity of each is 1 m.g.d.

The underdrainage system is of the conventional perforated-pipe type. It consists of 3-in. cast-iron pipe laterals, 6 in. on centers, connected to a central concrete conduit. The pipes are perforated with two rows of ¼-in.

holes, 6 in. on centers, staggered, and pointing down on a line 22½ deg. from the vertical.

There are four layers of graded gravel with a total depth of 18 in. above the floor. The characteristics of these layers are as shown in Table 1.

TABLE 1.—SPECIFIED THICKNESS AND SIZE OF GRAVEL LAYERS IN FILTERS.

Layer.	Depth. Inches.	Size of Opening through which Gravel Will Pass.	Size of Opening on which Gravel Will Be Retained.
Bottom	7	2½-in. circular ring	1½-in. circular ring
2nd	4	1½-in. circular ring	¾-in. square mesh
3rd	4	¾-in. square mesh	¼-in. square mesh
4th	3	¼-in. square mesh	No. 12 mesh U. S. Standard Sieve

Above the gravel is a 30-in. layer of Plum Island sand. The average of the analyses made of the sand before placing in the filters showed it to have an effective size of 0.35 m.m. and a uniformity coefficient of 1.48. The normal depth of water over the sand is 5 ft. There are three steel wash-water troughs in each filter, the top edges of which are 2 ft. 6 in. above the sand.

The filters discharge through rate of flow controllers to the common effluent main leading to the filtered-water reservoir. All valves are operated hydraulically and controlled from individual operating tables. Indicating loss-of-head and recording rate-of-flow gages are also located on the tables.

Wash Water. Wash water is obtained by direct pumping from the filtered-water reservoir. Duplicate pumping units are provided, the discharge of which passes through a rate of flow controller. The controller has a capacity range from 3 m.g.d. to 12 m.g.d., equivalent to a filter wash rate of 9 to 36 in. vertical rise per minute. The wash water effluent from the beds is discharged into the lagoons.

Filtered Water Reservoir. The filtered water reservoir is of concrete groined-arch construction, and has a capacity of 4 m.g. The water level is recorded in the office by means of a diaphragm-type liquid-level recorder.

From the reservoir, a 30-in. and a 24-in. main lead to the high-lift pumping stations of Salem and Beverly respectively.

Chemical Feed. There are four dry-feed machines, two for handling alum, and two for handling either hydrated lime, soda ash or carbon. Two manually-controlled solution-feed chlorinators are available for either pre- or post-chlorination. A manually-controlled direct-feed ammoniator is also provided.

Space for the storage of 60 tons of chemicals is provided on the second floor of the filter house. The dry chemicals are discharged from here by gravity through hoppers to the feed machines on the first floor. The

chlorine and ammonia machines are located in a separate room on the chemical storage floor.

Miscellaneous. The plant is provided with its own domestic water system. Duplicate vacuum pumps are installed for the domestic water pumps and wash water pumps. The recording instruments of the raw and wash-water Venturi meters, as well as the remote-control switches for the low-lift pumps, are located in the office. The laboratory is fully equipped for carrying out the necessary routine chemical and biological analyses.

Coagulation.

It has not been difficult to obtain proper coagulation of the raw water. During ordinary conditions a sufficient sized floc is formed in five minutes. Even when the lake water contains fresh coloring matter, not over ten minutes are required for proper coagulation.

Under these conditions, a good floc is obtained by running the mixing tanks in series, using only the stirring machine of the first tank. The rotational speed of the paddles is varied from 1 to 2 r.p.m., depending upon the ease of coagulation. The chemicals are added in the first tank where they are sufficiently well mixed to build up a moderately sized floc. The entrance to the second tank is near one corner, thus a rotary motion is imparted to the water. This serves to further floc formation, usually doubling or trebling its size over that in the first tank. The order of flow through the tanks is changed each week.

Use of Alum. Alum has been the only coagulant used. It has given good results in the treatment of waters of color varying from 24 to 70 p.p.m.

The pH values for good coagulation are fairly broad, varying from 5.4 to 6.7. Because of this, the natural alkalinity of the raw water has been sufficient to make unnecessary the addition of an alkali.

Results at present indicate that the dosage required is in nearly direct proportion to the color of the raw water. The amounts of alum applied have varied from 158 lb. per m.g., equivalent to 1.10 grains per gal., to 255 lb. per m.g., equivalent to 1.78 grains per gal. The average for the year was 210 lb. per m.g., equivalent to 1.47 grains per gal.

Use of Chlorine. In the control of tastes and odors, prechlorination was used for several weeks. The results of the treatment for this purpose will be discussed later in this paper. It was noticed, however, that pre-chlorination resulted in larger floc formation, enabling the cutting of the alum dosage by several per cent. Further work will be done to determine possible savings that might be made through the use of pre-chlorination as a standard practice.

Use of Activated Carbon. Activated carbon was also used a great deal for the control of tastes and odors. Contrary to the results obtained at some plants, activated carbon did not act as an aid to coagulation. In fact it appeared to retard it to some extent.

Basin Cleaning. The coagulation basins were cleaned three times during the year: in March, July and October. At the time of the first two cleanings there were approximately 7 ft. of sludge near the entrance of the basins which gradually decreased to 4 ft. at the end of the basins. This amount is apparently near the maximum that can be present for efficient operation. After cleaning, no measurable difference in odor of the settled water was observed, but a slight increase in filter runs was noticed. The cleaning in October was done to alleviate odor troubles originating from decomposed microscopic organisms, even though a maximum of only 5 ft. of sludge was present.

Sludge Lagoons. At the present time no definite conclusion regarding the behavior of the lagoons can be made, although the indications to date are that they will be satisfactory. As before mentioned, the third lagoon has not as yet filled; thus the quality of its effluent is unknown. It can be said, however, that the quality of the water in the second lagoon is far superior to that in the first; in appearance it has been better than the lake water itself.

There have also been only three discharges of sludge from the coagulation basins. No unusual conditions were noted after the first two discharges, but some difficulty was experienced after the third. A large portion of the sludge rose from the bottom of the first lagoon and floated in large masses on the water surface. These masses later consolidated and sank.

On a few occasions, odors arising from the lagoons were noticeable, especially at the time of the previously mentioned sludge trouble. Large numbers of algae or surface growths, however, have not appeared.

Filter Operation.

As stated previously, the average analysis of the filter sand before placing gave an effective size of 0.35 m.m. and a uniformity coefficient of 1.48. On November 23, 1936, slightly more than a year after placing, samples were collected from each of the eight beds. The results of analyses are tabulated below.

Table 2.—Mechanical Analyses of Filter Sand.

Date of Collection.	Filter No.	Effective Size. m.m.	Uniformity Coefficient.
Aug. 7, 1935	Before placing	0.35	1.48
Nov. 23, 1936	1	0.37	1.50
	2	0.38	1.47
	3	0.39	1.37
	4	0.40	1.46
	5	0.37	1.49
	6	0.39	1.43
	7	0.36	1.46
	8	0.37	1.41
Average on Nov. 23, 1936		0.379	1.449

These results show only a slight change in the characteristics of the sand during the year's operation. In general there is a slight increase in effective size and a slight decrease in uniformity coefficient.

Rates of Filtration. During the greater part of the year, water was filtered at the rate of 125 m.g.a.d. In order to increase the plant capacity during the months of May through August, the rate was stepped up to 140 m.g.a.d. This resulted in reducing the detention period in the coagulation basins from 4 to 3.56 hr. No difference was noted in the quality of the effluent at the higher rate.

Length of Runs. The average length of runs was only 15.3 hr. This was due to several causes. First, a good deal of investigation was done to determine the optimum quality of the settled water; this resulted in non-uniform loads being placed on the filters. Second, the filter sand is somewhat fine. Third, the short runs in May, July and October may be attributed to the presence of microörganisms.

In an effort to find a means of increasing filter runs, the point of application of lime for corrective treatment was changed on December 1, from the filter-effluent main to the filter-influent main. This treatment is still in the experimental stage, but it is believed that it will lead to a betterment in length of runs. Despite a deterioration in the quality of the raw water in December over that in November, an 18-per cent. increase in length of runs was obtained in December with this treatment.

This increase is probably due to several causes. The theory on which this trial was made was that the sand would acquire a lime coating, thus increasing the size of the grains. Another possible action is a breakdown in the size of the floc coming from the coagulation basin. This smaller floc would have a tendency to be carried down into the bed, rather than to be deposited mainly on the top surface. A much longer period would therefore be required to build up a solid mat. Up to the present time, no cementing of the sand grains or other deterioration of the bed has been noticed. This treatment will be further discussed later in this paper.

Washing. The average initial loss of head was 1.56 ft. Beds were generally washed at 8-ft. loss of head; the average of the year was 8.05 ft. A washing period varying from 3 to 5 minutes is required; the length depending upon the load placed upon the filters.

TABLE 3.— SAND EXPANSION AT 37° F.

Wash Rate. (Vert. in.— min.)	Per cent. Expansion. (By Tests.)	Per cent Expansion. (By Hazen.)
24	45.0	46.3
26	50.3	51.5
28	55.7	56.7
30	61.0	62.0
32	66.3	67.2
34	71.7	72.4
36	77.0	77.6

Wash-water velocities used varied according to the temperature of the water. Rates from 28 to 36 vertical inches per minute have been used. This gives an expansion of approximately 55 per cent. Actual tests of sand expansion have shown an extremely close correlation to the theoretical values obtained from Hazen's formula. This is illustrated in Table 3.

The percentage of wash water is relatively high. The average for the year was 4.6 per cent. This is due partly to the short filter runs, partly to the long period required for thorough washing, and partly to the large amounts used during adjustments of equipment.

Special Treatment of Beds.

The ease of coagulating Wenham Lake water has been of considerable help in diminishing filter-bed troubles. The floc reaching the filters is coarse and sticky, resulting in its removal at or near the surface of the filter. Possible bad effects resulting from the floc being drawn deeply into the bed are thus absent.

Sand Shrinkage, Cracking, and Mudballs. There has been little trouble experienced with sand shrinkage and cracking. On one or two occasions a very slight pulling away from the side walls has been noticed. This, however, has never exceeded $\frac{1}{16}$ in.

Mudballs have been present at various times, but have never been of large enough size or sufficient in numbers to cause any trouble. As yet their appearance has not been associated with any particular type of water, and seldom have they appeared in all the beds at the same time. They have only been present on the top of the sand, never below the surface. Because of their relative unimportance, no treatment such as the application to the bed of caustic soda has been used for their elimination. A simple light scraping of the top surface of the bed has sufficed.

Bacteria. A short time after the plant was put in commission, trouble was experienced with high 20-°C. bacterial counts in the filter effluent. These bacteria were easily killed by the subsequent chlorine treatment and were typical of those sometimes found in deep well supplies. Examination proved that their source was in the filter beds. Samples of the sand were analysed, but the grains were clean and devoid of any coating. An examination of the filter walls disclosed a fairly thick slime in many places. In view of this, the beds were treated with a strong dose of hypochlorite. The results of this were negative; it neither reduced the numbers of bacteria nor affected the slime. The application of copper sulphate was next tried with the same results. Recourse was then taken to the scrubbing of the wall surface above the sand with stiff brushes, followed by heavy hypochlorite treatment. A marked betterment in condition was at once observed, and after several washings of the beds at an increased rate of flow, this trouble disappeared. Since then there have neither been further slime deposits on the walls nor further trouble from bacteria.

Corrective Treatment.

It was the original intention to use soda ash for corrective treatment. It soon became apparent, however, that because of its low reactive power another dry feed machine would be needed or lime would have to be substituted. In view of the economy of lime, it was decided to substitute this for soda ash. Since then, except for a short period from January 28 to March 3, 1936, when soda ash was used, lime has been employed for corrective treatment.

The return to soda ash in January was occasioned by a complaint received from the Salem Gas and Electric Company. Coincident with the transition period from untreated to filtered water, the company experienced difficulty with their boiler-feed equipment. Their pipe lines had become badly clogged with a soft scale. This they attributed to the use of lime at the filter plant. Conferences were held with representatives of the light company, as a result of which a temporary return was made to soda ash. A careful investigation was then begun, including an inspection of the power plant and many mineral analyses of water and scale. The result of this indicated that the light company's troubles were due in part to an apparent sloughing off of distribution-system pipe-deposits during this transition period. These deposits had evidently precipitated in their boiler-feed pipes. As no evidence could be found to correlate the company's troubles with the use of lime at the filter plant, lime treatment was then resumed. No further complaints have since been received.

The present practice of corrective treatment has been to add sufficient alkali practically to neutralize the carbon dioxide. This has been accomplished by the application of 62 to 119 lb. per m.g. of lime. During the period of lime treatment the final effluent has had an average carbon dioxide content of 0.45 p.p.m., an average alkalinity of 22 p.p.m., and pH values ranging from 7.4 to 8.4. The use of lime has increased the average hardness of the water from 36 p.p.m. to 44 p.p.m.

The change in the point of application of the lime has introduced several problems on which research is now being carried on. It has been observed that there is a marked lag between changes in lime dosages and the results of such changes taking effect in the filtered water. This is evidently due either to the solution of or to the deposition of excess lime on the sand grains. In addition, if the pH value of the water is carried very far into the alkaline range, considerable color is redissolved and passes through the filters.

Chlorination.

During the summer of 1935, tap samples collected throughout both city distribution systems had given frequent positive presumptive tests. Investigation had clearly shown that the origin of the bacteria was the slime coating within the mains of the distribution systems. In addition, Beverly has an uncovered reservoir to which contamination had been traced.

This situation became sufficiently serious in July of that year to make necessary the installation of chlorinators in the pumping stations of Salem and Beverly. The use of chlorine resulted in the delivery from the stations of a nearly sterile water. The high chlorine demand of the lake water, together with that of the pipe slime, made it impossible to carry a chlorine residual. The bacteria, therefore, still thrived in the distribution systems.

With the opening of the plant it, therefore, became imperative to attack this problen with a strong ammonia-chlorine treatment of the filtered water. The initial treatment was at the rate of 5 lb. per m.g. of chlorine and of 1.25 lb. per m.g. of ammonia, or a 4 to 1 ratio. This was equivalent to a residual of 0.30 p.p.m. chlorine at the plant. Within a few days the presence of a chlorine residual was noted at various taps in the cities. As a result of experimentation to determine the best chlorine-ammonia ratio, a ratio of 2 to 1 was later adopted.

The results of this treatment were gratifying. Within two weeks, residual chlorine was present in all the large mains, and taps served by these mains were free from pollutional-type bacteria. The dead-end sections of the distribution systems responded more slowly to this treatment. With the aid of several flushings of these mains, however, chlorine residuals were soon obtained. Within six weeks, tests failed to show the presence of members of the coli-aërogenes group. There have been no positive tests since that time.

The quantities of chlorine and ammonia applied have since been reduced; at the present time the rate of chlorine application is 1.5 lb. per m.g., equivalent to 1.8 p.p.m., and that of ammonia is 0.75 lb. per m.g., equivalent to 0.9 p.p.m. This quantity is sufficient to give a residual of 0.01 p.p.m. chlorine in the most remote sections of the cities.

Odor Troubles.

Odor troubles have been of two kinds; those caused by organic matter, and those caused by microörganisms.

Organic Matter. The use of carbon applied to the mixing tanks has been successful in the treatment of organic odors. The doses used have varied from 1 to 3 p.p.m., depending upon the intensity of the odor. It has been found that best results have been obtained when the carbon is introduced into the second of the two mixing tanks. Both stirring machines are then used. This procedure apparently acts to prevent the enmeshing of carbon within the nucleus of the floc wherein it would be unavailable for odor removal.

Microörganisms. Microörganisms were more or less prevalent from May through November. On two occasions their numbers became so large that the lake was treated with copper sulphate. The first treatment, for *Anabaena*, was in July; the second, for *Asterionella*, was in November.

Carbon treatment, alone, for the removal of odors due to microscopic organisms, was not entirely satisfactory. Prechlorination was next tried,

TABLE 4.—SUMMARY OF OPERATION DAT

Month.	Quantity of Water — m.g.			Filter Operation.			
	From Lake.	Wash Water.	Net Yield.	Per cent. Wash Water	Av. Length of Runs.	Av. Loss of Head — ft.	
					Hrs.	Initial.	Final.
January	171.36	7.14	164.22	4.2	16.5	1.9	8.0
February	173.93	7.93	166.00	4.6	15.9	1.9	8.0
March	176.95	6.97	169.15[1]	3.9	15.4	2.1	7.9
April	165.53	5.83	159.70	3.5	15.1	1.7	7.8
May	188.45	10.56	177.89	5.5	13.1	1.5	8.0
June	193.59	7.22	186.37	3.7	15.1	1.4	7.9
July	208.05	9.31	197.44[2]	4.6	13.9	1.4	8.0
August	197.89	8.42	189.47	4.3	15.9	1.4	7.9
September	175.88	8.47	167.41	4.8	15.8	1.3	8.4
October	182.27	10.16	170.91[3]	5.5	13.7	1.2	8.3
November	171.18	9.37	161.81	5.4	15.1	1.4	8.2
December	172.17	7.89	164.28	4.6	18.3	1.5	8.0
Totals	2 177.25	99.27	2 074.65				
Averages per day per m.g.	5.95	0.27	5.67	4.6	15.3	1.56	8.05

[1] 0.83 m.g. from settling basins.
[2] 1.30 m.g. from settling basins.
[3] 1.20 m.g. from settling basins.

TABLE 5.—SUMMARY OF PHYSICA

JANUARY 1, 1936

Month.	Color — p.p.m.			pH Value.			Carbon Dioxide — p.p.m		
	Raw.	Settled	Final Eff.	Raw.	Settled.	Final Eff.	Raw.	Settled.	Final Eff.
January	52	19	6	7.0	6.0	7.2	5	15	3
February	53	27	8	6.7	5.8	6.8	8	17	8
March	52	29	9	6.6	5.8	7.1	9	17	5
April	44	16	5	6.9	5.7	7.4	3	10	1
May	44	16	6	7.0	5.8	7.4	3	10	1
June	35	16	7	6.9	5.8	7.4	5	11	0
July	40	15	10	6.8	5.8	7.5	6	12	0
August	36	15	10	6.8	6.0	7.5	5	11	0
September	31	11	8	7.2	6.2	7.9	3	9	0
October	33	12	6	7.3	6.4	8.7	2	9	0
November	41	22	8	7.2	6.6	8.1	2	8	1
December	48	22	8	7.1	6.3	7.9	3	9	1
Average	42	18	8				5	12	2
Maximum	53	29	10	7.3	6.6	8.7	9	17	8
Minimum	31	11	5	6.6	5.7	6.8	2	8	0

*Most probable numbers (Standard Methods, 1936, p. 220.)

JANUARY 1, 1936 TO DECEMBER 31, 1936.

CHEMICALS APPLIED — LB.							Electricity. k.w.h.
Alum.	Lime.	Carbon.	Chlorine.		Ammonia.	Soda Ash.	
			Pre.	Post.			
43 600	14 950	4 520	....	839	416	3 600	39 200
40 000	....	105	....	807	402	28 350	38 800
37 800	13 500	....	....	660	319	1 600	38 800
34 000	14 150	....	....	523	263	....	33 600
35 800	17 800	4 135	257	281	140	....	40 000
37 800	18 850	2 700	670	305	173	....	41 600
43 200	23 900	....	476	388	250	....	45 200
42 600	21 450	4 060	392	336	331	....	44 000
43 400	21 000	4 135	1 010	340	172	....	41 600
39 600	16 100	2 600	146	345	168	....	44 800
27 200	10 550	1 850	....	342	161	....	42 800
32 600	14 050	....	....	344	165	....	42 800
457 600	186 300	24 105	2 953	5 510	2 900	33 550	493 200
210	87[4]	27[5]	5.4[5]	2.6	1.3	167[5]	227

[4] Excludes time soda ash was used.
[5] For time used only.

CHEMICAL AND BIOLOGICAL ANALYSES.
DECEMBER 31, 1936.

ALKALINITY — p.p.m.			Free Chlorine. — p.p.m.	AGAR AT 20° C.		AGAR AT 37°C.		PRESUMPTIVE COLI-AEROGENES PER 100 C.C.*		
Raw.	Settled.	Final Eff.		Raw.	Final Eff.	Raw.	Final Eff.	Raw.	Filtered.	Final Eff.
21	12	27	.21	234	0	24	0	40	0	0
18	9	30	.24	52	0	20	0	27	0	0
18	11	23	.22	150	0	20	1	12	0	0
16	8	24	.21	53	0	16	1	21	0	0
16	9	24	.12	15	0	7	0	4	0	0
19	9	25	.15	38	0	9	1	18	0	0
20	10	30	.16	108	0	1	0	29	0	0
22	12	32	.15	30	0	21	0	9	0	0
23	11	32	.18	246	3	209	2	7	0	0.3
26	15	31	.17	24	0	9	0	8	0	0.1
28	22	32	.17	27	0	4	1	8	0	0
26	17	29	.15	440	0	31	1	53	1	0
21	12	28	.19	116	0	36	1	20	0.08	0.03
28	22	32	.24	440	3	209	2	53	1.00	0.30
16	8	23	.12	15	0	4	0	4	0.00	0.00

without carbon treatment. This was successful in killing the microörganisms. It, however, presented the problem of the removal of odors resulting from the liberation of the essential oils of the organisms as well as their odors of decomposition.

Carbon treatment in conjunction with prechlorination was then tried, both chemicals being added in the mixing tanks. Using a dose of 5.5 lb. per m.g. of chlorine and 20 lb. per m.g. of activated carbon, a successful treatment was obtained.

Results of Purification.

In order to obtain a better understanding of the results of purification, the monthly averages of the daily operation data and of the daily laboratory analyses have been compiled. These may be found in Tables 4 and 5.

Physical Results. The turbidity of the lake water averages less than 5 p.p.m., and presents no problem. The average color of the raw water was 42 p.p.m., and that delivered to the cities was 8 p.p.m., a reduction of 81 per cent. It is planned to keep the color of the filtered water between 5 and 10 p.p.m.

Chemical Analyses. The pH value of the raw water ranged from 6.6 to 7.3, and that of the final effluent from 6.8 to 8.4. The low pH values of the final effluent from January through March were due to the inability of adding sufficient soda ash. Carbon dioxide in the raw water averaged 5 p.p.m. and that in the final effluent averaged 2 p.p.m. During the period of lime treatment the carbon dioxide in the delivered water averaged 0.45 p.p.m. The amount of residual chlorine carried in the final effluent was high during the first part of the year, averaging 0.22 p.p.m., but this was reduced to an average of 0.15 p.p.m. chlorine after the distribution systems were cleared of bacteria.

Bacteria. The efficiency of bacterial removal is shown by a comparison of the analyses of raw and delivered waters. The average total bacterial counts on agar at 20°C. for 48 hr. for the raw and delivered waters were 116 and zero per c.c., respectively. The average counts on agar at 37°C. for 24 hr. were 36 and 1 per c.c., respectively. The average of the "most probable numbers"* of members of the coli-aërogenes group was 20 and 0.03 per 100 c.c. in the raw and delivered water, respectively.

Cost of Operation.

As is usual during the first year of operation, the costs are higher than may be expected in the future. Cognizance should be taken of this, therefore, in the comparison of the cost figures contained in Tables 6 and 7. Table 7 has been arranged to show the six principle items of expense.

The figures in these tables are computed for the period from January 1 to December 31, 1936 inclusive. Items chargeable to tuning up of equipment are not included in these figures, as it is the intention to show the

*"Standard Methods of Water Analysis," 1936 Ed., p. 220.

TABLE 6.— COST OF CHEMICALS FOR 1936.

Chemical.	Tons Used.	Cost—Ton.	Total Cost.	Cost—m.g. Yearly Basis.	Cost—m.g. When Used.
Alum	228.800	$ 26.93	$6 161.13	$2.83	$2.83
Lime	93.150	11.65	1 085.20	.50	.51
Soda Ash	16.775	27.92	468.33	.21	2.33
Carbon	12.052	98.01	1 181.27	.54	1.33
Chlorine (Post)	2.755	148.50	409.12	.19	.19
Chlorine (Pre)	1.477	148.50	219.26	.11	.41
Ammonia	1.450	310.00	449.50	.21	.21
Totals	356.459		$9 973.81	4.59	

TABLE 7.— OPERATING COST DATA.

Item.	Total Cost.	Cost — m.g.
Chemicals	$ 9 973.81	$ 4.59
Labor, Operators, Laborers	6 565.29	3.02
Laboratory, Chemists, Supplies	3 145.68	1.45
Power, Light	10 056.96	4.62
Fuel	1 576.47	0.72
Equipment, Service, Supplies	1 390.20	0.64
Totals	$32 708.41	$15.04

normal operating costs. The costs for chemicals and power are based on actual consumption at their unit costs. Other items are based on actual expenditures. No part of the fixed charges of the plant is included.

ACKNOWLEDGMENT.

I wish to acknowledge the many helpful suggestions tendered by Messrs. E. Sherman Chase and Stuart E. Coburn. Their help has been most valuable in the solving of many perplexing problems to which they have willingly given their time.

A word must be said for the excellent coöperation given by the filter operators during the trying tuning-up period of the first year. All inexperienced men to begin with, they have quickly developed into high-class operators. Their interest has been sincere and their many suggestions have been helpful.

The Salem and Beverly Filtration Plant is under the control of the Salem and Beverly Water Supply Board. Its personnel consists of Messrs. Charles Ross, Chairman, Frank P. Morse, and James W. Blackmer. The direct responsibility for the operation of the plant is vested in the Superintending Chemist, a position held by the writer.

ACTIVATED CARBON.

The Water Purifier.

BY E. A. SIGWORTH.*

[*Read September 24, 1936.*]

Carbon in the form of charcoal was used many years ago as a means of purifying water. Due to the inefficiency of the charcoal, however, this method of water purification was practically forgotton. With the advent of activated carbon, the application of carbon to water purification was revived. The reason for this revival can be more readily understood when we consider that some commercial qualities of activated carbon are as much as 1 000 times more effective in purification than ordinary charcoal.

The action of carbon in purification is dependent upon the surface attraction of the carbon particle for impurities, and the theory behind activated carbon is that the surface of the carbon particle has been tremendously increased, thus increasing its ability to absorb impurities. This increased surface is more internal than external and consequently each small particle of activated carbon would resemble, under a very powerful microscope, a small black sponge.

Activated carbon was first applied in water treatment in a granular form, whereby the water was allowed to percolate through a layer of the material. Activated carbon applied in this way was very effective as a water purifier, but progress was very slow due to the expense of the material itself and the fact that very expensive equipment was necessary for its utilization. Activated carbon in powdered form was first applied in 1930 at the New Milford, N. J., Plant of the Hackensack Water Company, under the supervision of George R. Spalding. The results obtained in this first test were so successful that during that year fourteen additional water plants tried activated carbon in powdered form. There are now over 800 water plants utilizing this method of application with successful results.

The results to be accomplished by the application of activated carbon are principally elimination of tastes and odors. These may be characterized as earthy, woody, vegetable, bitter, aromatic, fishy, marshy, swampy, muddy, chlorine and the like; and may originate from decaying vegetation, algae, industrial trade wastes, decomposing sludge, etc. The application of activated carbon in powdered form has successfully corrected these conditions together with many others, too numerous to mention and today is considered routine operation when tastes and odors occur.

Water plant operators have obtained other advantages which are worthy of mention at this point. Many cases have been reported where a much better alum floc is obtained. The activated carbon apparently acts

*Industrial Chemical Sales Company, Inc., New York, N. Y.

as a nucleus for floc formation, yielding a more satisfactory particle for settling. In some cases, the improvement has been so marked that it has been possible to reduce the amount of coagulant and still obtain a satisfactory floc. As a certain amount of the carbon settles out in the sludge in the sedimentation basin, further benefits from the application are derived. It acts as a stabilizer of the sludge, keeping it in sweeter condition and preventing decomposition with the resultant scum troubles. Due to the fact that the sludge is stabilized, the necessity for frequent cleanings of sedimentation basins is considerably reduced. H. E. Watson of Newport, R. I., reports that when carbon is present in the sedimentation sludge, it is possible to clean the basins much more easily and quickly, since the sludge does not pack so densely as when activated carbon is not present. The efficiency of the filters is in most cases considerably increased as longer filter runs are generally reported.

In the designing of new plants, it is quite often possible to show considerable savings if aëration of the water has been contemplated. One of our foremost consulting engineers in the water purification field advises that he is recommending the use of activated carbon in place of aëration. By eliminating aëration in a water plant, it is possible to save not only the cost of the aërating equipment but also the cost of pumping equipment, since smaller units would be necessary. Further savings would be accomplished during those periods of the year when aëration would not be employed, since the plant would be operating very near the maximum rather than the reduced load with resultant reduced pumping efficiency. I believe that if all these factors were taken into consideration, the use of activated carbon would actually show a monetary saving to the majority of water plants.

Water plant operators who are only troubled occasionally with tastes and odors, are finding it advantageous to apply small amounts of carbon continuously. It has been found that taste and odor troubles occur at much less frequent intervals, and that, when they do occur, they are not as severe and therefore require less carbon to actually eliminate the trouble.

The importance of feeding sufficient activated carbon to water so that a palatable effluent is supplied at all times cannot be stressed too highly. The plant operator or superintendent should consider not only the present user but also the potential user of his supply. Many towns have lost prospective manufacturing plants, because they have been judged by just one taste of the water. Quite a number of water works men have therefore adopted the very excellent procedure of obtaining the reactions of strangers to the taste of the water. These men realize that their senses of taste and smell may become accustomed to the particular water and that a stranger will be the first to detect trouble. I know several water works men who make it a practice to visit hotels periodically and question guests regarding the palatability of the water.

The method of application of activated carbon to water is r simple. Since the action of activated carbon is dependent upon the of its surface to the impurities to be removed, it is essential that each be thoroughly wet with water in order to accomplish most efficient In applying carbon to water, it is therefore advisable to feed it th dry-feed machine equipped with a water ejector or to mix it thoroug water if solution-feed is employed. The application of activated car dry-feed machine is preferable as considerably less attention is on the part of the operator. Solution methods of feeding are also en but in these cases it should be remembered that the carbon must oughly saturated with the water and also that a mixer should be i in the solution tank in order to keep the activated carbon sufficie persed in the water. The carbon in suspension is led from the equipment to the point of application, which may be at almost a prior to filtration.

The particular point of application in the water works syst depend somewhat on actual operating conditions. A recent survey of activated carbon, made by "Public Works," gives some interesti It was found that of 96 plants reporting on actual points of app 61 were feeding prior to sedimentation, 27 after sedimentation a split treatment. By application prior to sedimentation we mean app to the raw water, in the mixing chamber or directly into the sedim basin. By application after sedimentation, we mean applicatio filter influent, directly on top of the filters or in the filter back-was By split treatment we mean that activated carbon is being fed bot and after sedimentation. Conclusions to be reached from this su that the most generally satisfactory point of application is prior mentation. No doubt the reason for the preference for this particul of application is due to the many advantages other than taste a removal which may be accomplished such as improved coagulat bilization of sludge and so forth.

Further data obtained from this same survey shows that the tions of carbon actually to be applied will vary from a few pou high as 200 pounds per million gallons of water. The average of 9 reporting is approximately 16 pounds per million gallons. On the this average, the cost of treatment will only amount to approximat per million gallons of water.

If we consider all of the benefits to be derived from the appli activated carbon, together with the advantages of obtaining the of the consumer, reduction of complaints and reduced worry on of the plant superintendent, it is evident that the use of activate in any water plant having taste and odor troubles will pay its own w

The method of application of activated carbon to water is relatively simple. Since the action of activated carbon is dependent upon the exposure of its surface to the impurities to be removed, it is essential that each particle be thoroughly wet with water in order to accomplish most efficient results. In applying carbon to water, it is therefore advisable to feed it through a dry-feed machine equipped with a water ejector or to mix it thoroughly with water if solution-feed is employed. The application of activated carbon in a dry-feed machine is preferable as considerably less attention is required on the part of the operator. Solution methods of feeding are also employed, but in these cases it should be remembered that the carbon must be thoroughly saturated with the water and also that a mixer should be installed in the solution tank in order to keep the activated carbon sufficiently dispersed in the water. The carbon in suspension is led from the feeding equipment to the point of application, which may be at almost any point prior to filtration.

The particular point of application in the water works system will depend somewhat on actual operating conditions. A recent survey of users of activated carbon, made by "Public Works," gives some interesting data. It was found that of 96 plants reporting on actual points of application, 61 were feeding prior to sedimentation, 27 after sedimentation and 8 by split treatment. By application prior to sedimentation we mean application to the raw water, in the mixing chamber or directly into the sedimentation basin. By application after sedimentation, we mean application to the filter influent, directly on top of the filters or in the filter back-wash water. By split treatment we mean that activated carbon is being fed both before and after sedimentation. Conclusions to be reached from this survey are that the most generally satisfactory point of application is prior to sedimentation. No doubt the reason for the preference for this particular point of application is due to the many advantages other than taste and odor removal which may be accomplished such as improved coagulation, stabilization of sludge and so forth.

Further data obtained from this same survey shows that the proportions of carbon actually to be applied will vary from a few pounds to as high as 200 pounds per million gallons of water. The average of 96 plants reporting is approximately 16 pounds per million gallons. On the basis of this average, the cost of treatment will only amount to approximately $1.00 per million gallons of water.

If we consider all of the benefits to be derived from the application of activated carbon, together with the advantages of obtaining the good will of the consumer, reduction of complaints and reduced worry on the part of the plant superintendent, it is evident that the use of activated carbon in any water plant having taste and odor troubles will pay its own way.

MASTER METERS.

The G-Men of the Waterworks Industry.

BY HAROLD W. GRISWOLD.*

[*Read September 25, 1936.*]

Unrecorded or unknown use of water in the distribution system, which in some plants may amount to as much as 25 per cent., still offers a fertile field for study and ingenuity.

Illegitimate use can be controlled by rigid inspection and by equipping fire lines with detector meters, or with the so-called Baltimore Check and Bypass Meter, but the cost of either procedure is apt to be out of proportion to the benefit obtained. Under-registration of consumers' meters can be controlled to a considerable degree by proper maintenance, by frequent tests of the larger meters in place, and by proper sizing of meters from measurements of rates of use in the premises served.

For obtaining these rates of use from meters, the Water Bureau at Hartford has been using, for a number of years, a mechanical rate recorder invented by their Distribution Engineer, Frank S. Brainard, and put on the market a year or two ago under the name of "METER MASTER."

As an illustration of what can be done with this instrument to check under-registration of meters and to select the proper size and type of meter for each consumer, data on three investigations recently made in Hartford are submitted. Each study was made for securing information for answering the following questions:

1. Is the compound meter on this service of the proper size for maximum demand and for minimum loss of revenue?
2. Will more or will less water be registered on a battery of single meters having the same total capacity as the compound meter than by the compound meter now on the service?

The procedure in each case was to hook up a battery of three single meters in series with the compound meter, read each meter daily for at least a week and take rates with the recorder for a Saturday and a Sunday and for at least one week day. Assuming that five times the week-day consumption plus the Saturday and Sunday consumption gives a fair average consumption for a week, the percentage of weekly use at each rate can be obtained from the charts. By plotting this with the accuracy of the registration curve of the compound meter above and the accuracy of the registration curve of the battery of meters below, a good picture of what is happening can be obtained, and a little further study will show causes and suggest remedies.

*Deputy Manager and Deputy Chief Engineer, Water Bureau, Metropolitan District of Hartford County, Conn.

State Office Building. A five-story office building, about 150 by 360 ft. in plan, housing 1 135 persons, was the subject of the first test. The water-using equipment consists of 160 flushometer valves, 210 faucets, 16 fountains, 4 boilers, and several connections to lawn sprinklers, all served through a 6-in. service pipe, on which is a 3-in. compound meter. Six months' consumption for the premises averages 575 000 cu. ft., or an average of 3 160 cu. ft. per day.

The following data were obtained from the "Meter Master" charts from the 3-in. compound meter:

The small meter operates continuously.
The maximum rate per hour is 37 cu. ft.
The minimum rate per hour is 14 cu. ft.
The larger meter registers 76.5 per cent. of the total consumption and does not operate between 11 P.M. and 7 A.M.
Its maximum rate per hour is 450 cu. ft.
The consumption during the maximum hour was 465 cu. ft. and during the minimum hour, 21 cu. ft.
The percentage of total consumption used at various rates is as follows:
6.5 per cent. at rates below 0.4 cu. ft. per minute,
2.4 per cent. at rates between 0.4 and 0.8 cu. ft. per minute,
6.4 per cent. at rates between 0.8 and 1.65 cu. ft. per minute,
4.3 per cent. at rates between 1.65 and 3.3 cu. ft. per minute,
5.9 per cent. at rates between 3.3 and 5.0 cu. ft. per minute,
55.3 per cent. at rates between 5.0 and 6.6 cu. ft. per minute,
19.2 per cent. at rates between 6.6 and 7.8 cu. ft. per minute.
From the registration curve of the 3-in. compound meter, it appears that this meter is of the proper size for the demand, but is failing to register about 3 per cent. of the water passing through it. The battery of 1½-in. meters registers about 2 per cent. less than the compound meter.

Children's Village. This orphanage was chosen for the second test. It consists of eight two-story buildings, housing 132 children and adults. Water-using equipment consists of 25 toilets, 4 showers, 4 boilers, 3 water-cooled refrigerators, and a laundry, all served through a 4-in. service, on which there is a 3-in. compound meter. Six months' consumption is 261 000 cu. ft., or an average of 1 434 cu. ft. per day.

The following data were obtained from the "Meter Master" charts on the 3-in. compound meter:

The small meter operates continuously.
The maximum rate per hour is 52 cu. ft.
The minimum rate per hour is 6 cu. ft.
The large meter registers 55 per cent. of the total consumption and does not operate from 11 P.M. to 6 A.M. Its maximum rate per hour is 174 cu. ft.
Total consumption during the maximum hour is 209 cu. ft., and during the minimum hour, 6 cu. ft.
The percentage of the total consumption used at various rates is as follows:
3.2 per cent. at rates below 0.17 cu. ft. per minute,
3.3 per cent. at rates between 0.17 and 0.41 cu. ft. per minute,
10.4 per cent. at rates between 0.41 and 0.85 cu. ft. per minute,
32.2 per cent. at rates between 0.85 and 1.65 cu. ft. per minute,

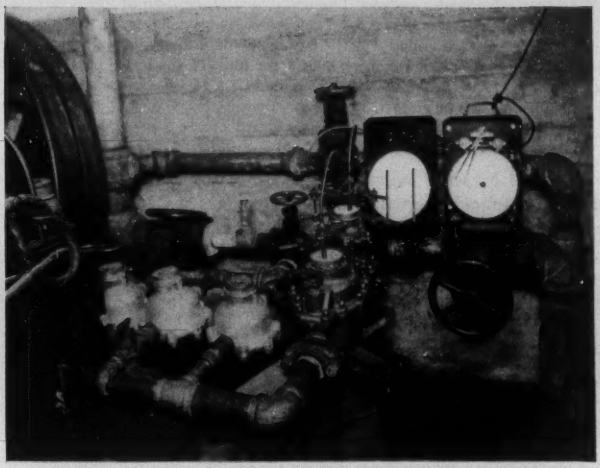


FIG. 1. — TWO-INCH COMPOUND METER, "METER-MASTER" AND BATTERY OF THREE ONE-INCH METERS AT HARTFORD EMPIRE COMPANY.

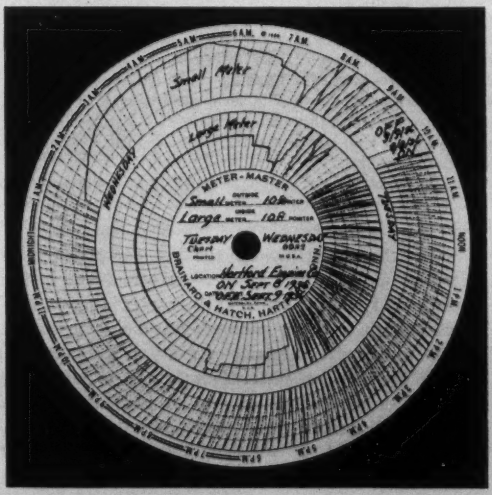


FIG. 2. — CHART FROM "METER MASTER" AT HARTFORD EMPIRE COMPANY.

23.6 per cent. at rates between 1.65 and 2.5 cu. ft. per minute,
27.3 per cent. at rates between 2.5 and 3.5 cu. ft. per minute.

From the registration curve of the 3-in. compound meter, it appears that, due to good registration at low flows, only about 1 per cent. of the water passing through it is unregistered. A 2-in. meter would be sufficient to deliver water at the maximum rate demanded at this location.

The battery of three 1½-in. meters registered nearly 5 per cent. less water than the compound meter.

Hartford Empire Company. This factory, selected for the third test, manufactures machinery and supplies for glass making. The building is about 75 by 300 ft. in plan, 1½ stories with basement. About 250 are employed. Water-using equipment consists of two water-cooled compressors, five toilets, two boilers, one hydraulic elevator, all served through a 4-in. service and 2-in. compound meter. Consumption for six months is 246 000 cu. ft., or an average of 1 350 cu. ft. per day.

The following data were obtained from the "Meter Master" charts from the 2-in. compound meter:

The small meter operates continuously but very slowly from 1 A.M. to 7.30 A.M Its maximum rate per hour is 78 cu. ft., and its minimum rate per hour, 1 cu. ft.

The large meter registers 41 per cent. of the total consumption and does not operate from 8 P.M. to 7 A.M. Its maximum rate per hour is 70 cu. ft.

The consumption during the maximum hour is 148 cu. ft., and during the minimum hour, 1 cu. ft.

The percentage of the total consumption used at various rates is as follows:
1.3 per cent. at rates below 0.18 cu. ft. per minute,
2.3 per cent. at rates between 0.18 and 0.40 cu. ft. per minute,
1.3 per cent. at rates between 0.40 and 0.50 cu. ft. per minute,
34.0 per cent. at rates between 1.0 and 1.15 cu. ft. per minute,
5.3 per cent. at rates between 1.5 and 1.6 cu. ft. per minute,
55.8 per cent. at rates between 2.1 and 2.5 cu. ft. per minute.

From the registration curve of the 2-in. compound meter, it appears that practically 100 per cent. of the water passing through is registered.

The battery of three 1-in. meters registers about 2½ per cent. less water than the compound meter.

A smaller meter might be used satisfactorily so far as the present demand is concerned but with a 4-in. service present practice would not sanction the installation of a meter smaller than 2-in.

Figure 1 shows the arrangement of the 2-in. compound meter, battery of 1-in. disc meters, and "Meter Master" rate recorder at the Hartford Empire Company. A typical chart from this meter is shown in Figure 2.

On the basis of these tests, it seems reasonable to conclude that a battery of disc meters having the same capacity as a compound meter will deliver from 2 to 5 per cent. less water than the compound meter, assuming that all meters are in good condition and adjustment, and that perhaps two out of three large meters could be replaced by those of smaller size and still meet consumer demand.

SOME FLOOD PRODUCING STORMS OF THE ATLANTIC SEABOARD.

BY MONTROSE W. HAYES* AND HORACE R. BYERS.†

[*Read September 23, 1936.*]

In estimating flood flows, particularly the probable maximum flows, it has always been customary to superimpose some particularly heavy rainstorm on the drainage area above the reach for which the estimate is being made. The storm may be transposed from a foreign basin, or, if it occurred in the basin under consideration, it may be shifted enough to place the heaviest rain in the position desired by the investigator. In the beginning little attention was paid to meteorologic characteristics, and it was assumed that rain of a given amount and intensity having occurred over one area could just as well fall in another, provided the two areas were not in radically different climatic zones. In recent years, considerable attention has been given to the weather map, and the barometric areas on the map have been moved one way or another, bringing about the transposition of a storm, or in reality the construction of a hypothetic storm.

Perhaps all investigators have realized the danger in this procedure and have been cautious in the use of the hypothetic rainfall, but as the concept of synoptic meteorology has undergone a marked change in recent years, and is now much more complex than it was when surface conditions only were considered, an opportunity is presented for the meteorologist to be of considerable aid to the hydrologist in making estimates of flood flows. Quite generally it will be admitted that a comprehensive study of floods must embrace flood causes, and as floods are the result of precipitation it is logical that the study should take into account the causes of precipitation.

Storm of May 30 to June 1, 1889. The three-day rainstorm of May 30 to June 1, 1889, has been selected for consideration because it was, and still is, the outstanding occurrence of its kind over the Potomac and Susquehanna basins, and perhaps the James Basin. It is well known, of course that it extended to the west of the Appalachian divide and gave high water in the headwaters of the Ohio, but had it not been for the disaster at Johnstown, Pa., caused by the failure of a dam, it would not have been impressed upon the public mind as a storm of importance in the drainage west of the mountains.

It is thought this flood, which came in late spring, may be considered a standard as representing run-off with the factors of a snow covering and frozen ground absent. Also, as the temperature and rainfall for a month

* Chief of the River and Flood Division of the U. S. Weather Bureau. (Died November 16, 1936.)
† Meteorologist, Meteorological Research Division of the U. S. Weather Bureau.

had been nearly equal to the usual average (both were only slightly deficient) it would seem the storm might be considered fairly representative of one with its run-off complicated to a minimum extent by previous wet or dry weather, or high or low temperature. However, as the vegetal covering was well advanced, but was not at its maximum stage, it seems obvious that there would have been a greater run-off from the same rainfall a month earlier, and a smaller run-off several weeks later.

Storm of November 2 to 4, 1927. October, 1927, was a wet month in New England and New York, but in the last 7 days of the month there was little rain and the weather was cool; temperatures below freezing were general on the 31st. The first day of November was unseasonably hot and was followed by torrential rains which began on the 2nd and continued into the 4th; the heaviest falls were on the 3rd.

There was no snow on the ground when the rains began and any freezing that might have been caused by the comparatively low temperatures of the last of October was slight, and doubtless complete thawing took place under the influence of the high temperatures on the first of November. Transpiration, of course, was at a minimum.

Several hydrologists have studied the run-off thoroughly and have published their conclusions, and it seems unnecessary to say more than that the meteorological aspects of the storm are interesting.

Storm of July 7 to 9, 1935. The excessive rains on July 7 to 9, 1935, were also confined to a relatively small area. In New York the fall was on July 7 and 8 and in Pennsylvania it was on July 8 and 9. The June rainfall in New York was more than the average. The departures as a rule were small but a few scattered places had as much as 1.5 to 2.5 in. in excess of the normal. In Pennsylvania it was about normal. In the first six days of July there was no rain of consequence in either state. The storm, or storms, might be considered of unusual interest owing to the fact that the excessive rain was in a measure spotty, but at the same time covered a sufficient portion of several basins to give damaging floods.

Storm of March 17 to 20, 1936. The storm of March 17 to 20, 1936, was remarkable in many respects. It appears to have given flood producing rains over an unprecedentedly large part of the Atlantic slope. It caused moderate floods in the rivers of North Carolina, while in the basins of the James and the other rivers to the northward, even as far north as northern New England, and in the headwaters of the Ohio, the floods were unprecedented. The height of these floods, however, was not due entirely to rainfall, but to the following contributory causes.

For a number of years the winters, in general, had been mild, especially the 5 up to and including 1934–35. In contrast, the winter of 1935–36 was one of the severest ever experienced in many parts of the country. The coldest weather occurred during a period of about 30 days ending around February 20. In addition to the severe cold the snowfall was the heaviest in many years in the country north of the Potomac and Ohio Rivers, and

late in February and early in March mild rainy weather caused rises and ice gorges in most of the rivers of New England and the Middle Atlantic States. In Virginia and Maryland some damage resulted from ice movement, but the combined run-off from the rain and melted snow was insufficient to give injurious floods. New England, New York, and Pennsylvania were not so fortunate, and there was considerable damage from both the crushing effects of ice and from flood waters.

By March 16, the streams in Virginia and Maryland had fallen considerably and a fall had begun north of Maryland. Conditions were approaching normal, but on March 17 a storm over south-central North Carolina was causing general rains over the Middle Atlantic Seaboard and the upper Ohio Basin. These rains were phenomenally heavy in the country to the west of the coastal plains and along the divide between the eastern rivers and the upper Ohio Basin. Amounts of nearly 5 in. occurred in 12 hrs. or less over a large area in the mountainous regions of Virginia, Maryland and western Pennsylvania, and more than 6 in. fell at some stations in a period of 48 hr. or less. The storm moved slowly in a northeasterly direction and the rains were heavy over all of the North Atlantic seaboard. To the northeastward the amounts were not so heavy, but from a flood-producing viewpoint the results were about the same, because the rivers in the southern part of the area under consideration were much lower at the beginning of the rain than they were in Pennsylvania, New York, and New England.

The rains fell on well-saturated and semi-frozen soil, or, in elevated regions, on a heavy blanket of snow of a high water content, and the percentage of run-off was unusually high. This, coupled with the fact that the northern rivers were at or above flood stage, and those in Maryland and Virginia, while not in flood were higher than normal, gave the most disastrous floods of record in the James, Potomac, Susquehanna, Connecticut, and Merrimack Rivers, in some of the tributaries of the Ohio River in Pennsylvania, and in the Ohio from Pittsburgh, Pa., to below Wheeling, W. Va.

The Ohio River flood gave a record crest-stage of 46.0 ft. at Pittsburgh on March 18. The previous high water record was 38.7 ft. on March 15, 1907.

All parts of the Atlantic Slope have had heavier rains than fell on March 17 to 20, 1936, but they were not caused by the same storm. Also the maximum falls have not, in any part of the Atlantic Slope north of North Carolina, occurred when conditions were as favorable for flood causation as in March, 1936.

METEOROLOGICAL CONDITIONS DURING THE MARCH 1936 AND OTHER NOTABLE FLOODS.

BY HORACE R. BYERS.*

[*Read September 23, 1936.*]

There are several fields of science and engineering which concern themselves with the water cycle of the earth and its atmosphere. Many of these fields overlap. For example, the evaporation of water from the oceans is a common problem of the oceanographer and the meteorologist; the evaporation from land areas is the concern of plant physiologists, soil experts, geologists and general hydrologists as well as meteorologists. Similarly, the meteorologist's role in surveys of run-off and floods is that of a collaborator with engineers and other scientists. In the study of one of the major phases of the water cycle — that accomplished in the atmosphere and involving the transport, accumulation and condensation of water vapor, the formation and distribution of precipitation — the meteorologist, however, has a unique position. It is his own special world, one might say.

I am here to speak to you as an ordinary weather man with my mind more in the clouds and the upper air currents than on the rivers, lakes and reservoirs which these clouds and upper air currents serve to maintain. Although I will speak about floods, and those of last spring in particular, I will consider them only from the viewpoint of that somewhat mysterious and essentially aërial phase of the water cycle which is the meteorologist's own. In the study of the mechanics of this phase of the cycle, the science of meteorology has progressed a long way in recent years since the publication, about fifteen years ago by the two Norwegians, J. Bjerknes and H. Solberg of the now famous paper "Meteorological Conditions for the Formation of Rain."

The methods developed principally by the Norwegians, and now generally called the "air-mass analysis" system, are based on an attempt to investigate the exact physical processes involved in the weather rather than by less direct correlations previously employed. Essentially, the air mass system is derived from the fact that over the earth there exist regions where the air acquires properties of moisture and temperature that are characteristic of the particular region. The reason for the existence of such distinct areas is the unequal distribution of the heat from the sun over the earth's surface. This results in regions that are dry and cold, as, for example, the northern interior of Canada in winter, or in warm, damp localities, such as the Gulf of Mexico and the Caribbean Sea. When air remains stagnant or moves for a long time over the Gulf of Mexico it becomes warm and moist, while the air over Canada becomes cooled by contact with the snow-

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covered surface. Similarly, the northern oceans produce other properties in the air in which we would recognize high moisture and moderately low temperatures.

These separate and distinct bodies of air which have acquired their properties in separate "source" regions are called air masses, and each is given a name according to the place of its source. Thus, we speak of Polar Canadian, Polar Pacific, Polar Atlantic, Tropical Pacific, Tropical Gulf and Tropical Atlantic air masses.

These air masses, on account of their differences in temperature, also have differences in density. The warm air will be lighter than the colder currents, as would be the case in all fluids and gases. This means that where two different air masses come together, there will be a discontinuity in density between them, but the surface of discontinuity cannot be a vertical one. The warm air, because it is lighter, is always seeking to climb up over the colder, heavier air, while the latter seeks a lower level.

These discontinuities which form the boundaries between two air masses of different densities are called *fronts*. When the movement of the currents is such that the colder air is advancing over territory formerly occupied by the warmer air, the front is called a *cold front*. If the warmer air is capturing territory formerly held by the cold air, the discontinuity becomes a *warm front*. A third type of front, formed when the cold front overtakes a warm front and displaces the warm surface air which lies between to some higher level, is called an *occluded front*. Cold fronts are often identical with what is called a squall line or wind-shift line.

Along the fronts or air mass boundaries, then, a lifting of the warmer air up the slope of the discontinuity surface or front is taking place. The rapid decrease in pressure with height causes this air to expand. This expansion cools the air in accordance with the physical principle that an expansion of a gas without any outward addition or subtraction of heat causes it to cool at a known rate. This cooling reduces the temperature of the rising air to the condensation point; i.e., the temperature at which the water vapor can no longer be contained in the invisible vapor form but changes to small liquid water drops. As this process continues, the drops grow large enough to fall out of the clouds as rain. At very low temperatures, the precipitation will be in the form of snow instead of rain, or, under certain special conditions, hail or sleet. Clouds may also form in the cold air over which the warm air is sliding, their formation depending on a wide variety of conditions within the cold air.

A further result of the interaction of the air masses along the fronts is the production of low pressure areas, which have their centers along the fronts. The low pressure centers and their movements have formed the basis for forecasting under the older methods. The air-mass system still considers the significance of the low barometer, but only as a result of the motions of the fronts. The low pressure arises along a boundary between two air masses, appearing first on the weather map as a horizontal wave

or inflection in the front. As the warm air from the south makes inroads into the cold northerly air mass, the latter pushes southward and a low pressure center is formed between them around which the air masses rotate in the usual counter-clockwise direction. Each low pressure area or cyclone, as it is called, goes through a definite growth and degeneration ending in "occlusion" when the cold front, which usually rotates round the center more rapidly than the warm front, overtakes the latter and displaces the air of the "warm sector" to higher levels.

Another important aspect to consider, especially in our eastern mountain regions, is the behavior of air masses in mountainous regions. We know from direct physical reasoning verified by observational evidence that the vertical distribution of temperature within the air determines whether it will resist vertical displacement in passing over the mountains or rise freely and produce heavy rain with the expansional cooling during the lifting. If the temperature decreases slowly with height, the air is described as stable and under such conditions it tends to flow around the mountains instead of rising over them. A rapid rate of temperature fall with altitude is characteristic of an unstable atmosphere which is easily set into vertical motion. Heating or cooling of air at the surface are the principal processes determining unstable or stable stratifications.

Johnstown Flood, 1889. Let us examine the meteorological conditions in some of the notable floods of the eastern United States, and suppose we start with the famous "Johnstown flood" of 1889. The rain in this case fell out of the Tropical Atlantic air mass which was moving rapidly against the mountains from the southeast. This flow is practically normal to the axes of the ridges, and since this air mass during the warm season is an unstable one, pronounced lifting over the mountains occurred to produce heavy rainfall. The cold front moving from the west brought in colder, drier air, but its movement was so slow that the streaming of the moist, unstable tropical air from the southeast prevailed for a long enough time to permit the accumulation of an unprecedented amount of rainfall. With the passage of the cold front, the rain stopped.

New York State Flood, 1935. This type of meteorological condition — the movement of unstable tropical Atlantic air from the southeast over the eastern mountain ranges — seems to be a typical producer of floods during the warm season. Almost an identical type of situation occurred in the July floods of 1935. In this case we had definite proof of the instability of the tropical Atlantic air in the form of meteorological airplane ascents at Mitchell Field, L. I., and at Lakehurst, N. J., on the morning of July 8. These data showed that a lifting of only 2 000 ft. would set off vertical currents in this air which could account for very heavy rains. This conclusion was arrived at from thermodynamical calculations.

New England Flood, 1927. The floods in New England during the first few days of November, 1927, were of the same type, but it seems to be stretching our point to call these "warm season" floods. However, the

stability or instability of the tropical Atlantic air is determined more by the seasons in the ocean rather than on the land, and we know that the seasons in the oceans are delayed a month or two after those of the land, so that September represents almost "mid-summer" conditions for the ocean waters and October can be considered as a "warm" month. Furthermore, snow and ice are not present over the land and if high temperatures have prevailed there for some time, as in the case of late October and early November of 1927, we have conditions approaching those of the warm season. In the case of these floods then, as also in the 1889 and 1935 storms, tropical Atlantic air forced upward by the mountains played a dominant role. In the 1927 case, however, there was more activity connected with the cold front than in the other two instances, and much of the rain, especially that around Burlington, Vt., seemed to be associated with the action along the slow-moving cold front.

The Johnstown flood and the New England flood of 1927 as well as the 1935 phenomenon represented exceedingly heavy precipitation. They occurred as the result of run-off of water provided by rain alone without any thawing being involved. The question that logically arises is whether or not we are justified in assuming that such a situation could occur during a time of year when a great amount of thawing would be involved, such as, for example, during the month of March. From considerations of the physical processes involved in the rainfall, we would answer in the negative. Again the question of stability or instability of the air mass is to be considered. Since we have found that the heaviest rainfall occurs in the tropical Atlantic air mass, this is the only one for which these properties need be studied. In March we find that tropical Atlantic air moving into our area from the southeast is subjected to a great amount of cooling from below as it moves northward and shoreward over the ocean. Under summer conditions there is very little cooling of the warm Gulf Stream air as it moves toward the land in the warm season. In addition to the ocean cooling effects, if warm air is moving over ground which is just thawing and which, therefore, has a temperature of 32°F., the cooling of the overlying warm air by contact with the cold surface imparts further stability.

New England Flood, 1936. In the March, 1936, storm there was a pronounced transport of tropical Atlantic air into the mountains, but very little precipitation fell from it on account of the stability produced by its being cooled from below by contact with the cold inshore ocean currents and the chilled earth. In this flood, ordinary precipitation at warm and cold fronts was involved. A third factor to be considered is that it is only an essentially summer or late spring condition that can keep tropical Atlantic air moving over the land for a long period of time. This requires a quasi-stationary center of high pressure over the ocean, a phenomenon which occurs only when the ocean waters are cold relative to the land — therefore, only in the warm season. Accordingly, it seems reasonable to

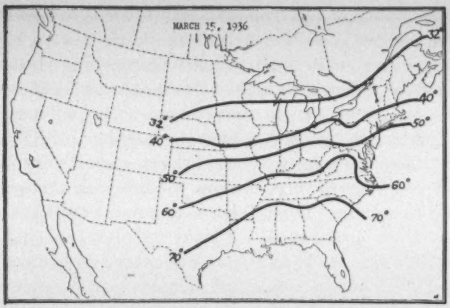

FIG. 1. — MEAN AIR TEMPERATURES (°F.)
MARCH 15, 1936.

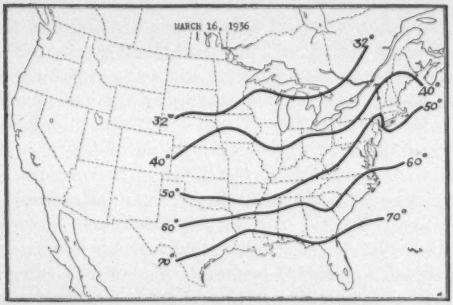

FIG. 2. — MEAN AIR TEMPERATURES (°F.),
MARCH 16, 1936.

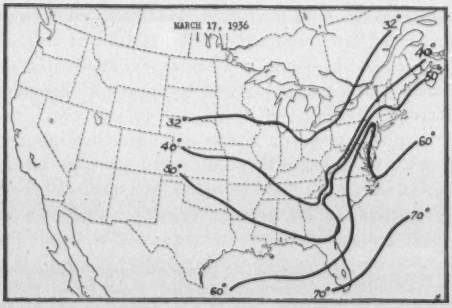

FIG. 3. — MEAN AIR TEMPERATURES (°F.),
MARCH 17, 1936.

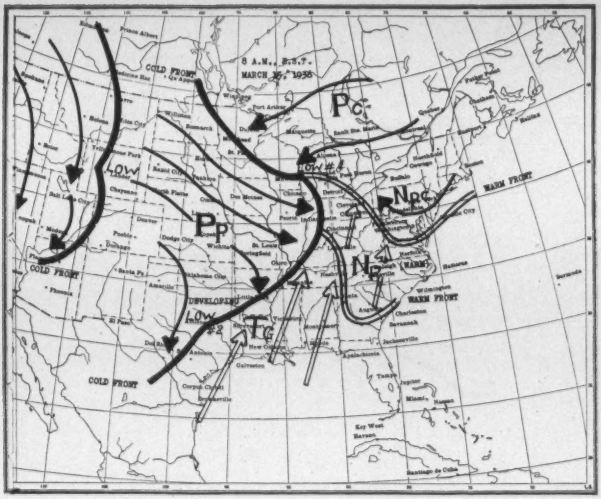

FIG. 4.—FRONTS, AIR WAVES AND AIR STREAMLINES, 8 A.M., MARCH 15, 1936.

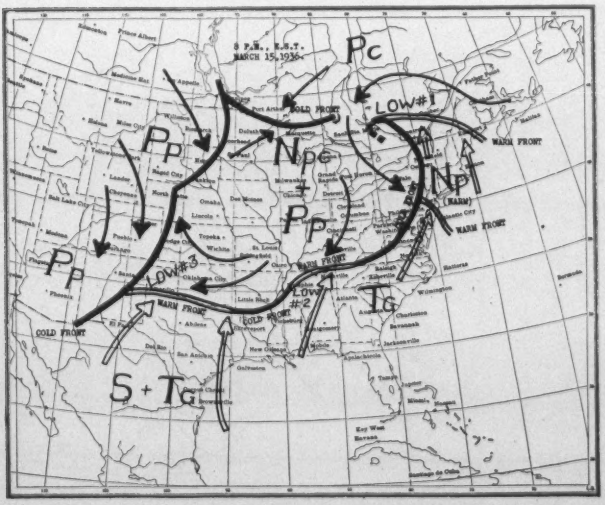

FIG. 5.—FRONTS, AIR WAVES AND AIR STREAMLINES, 8 P.M., MARCH 15, 1936.

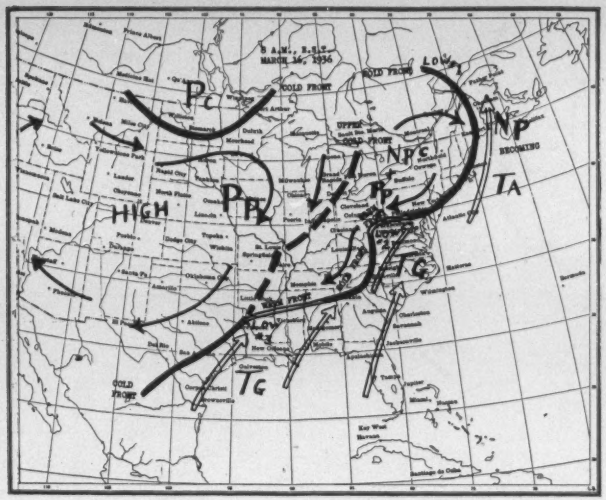

FIG. 6.—FRONTS, AIR WAVES AND AIR STREAMLINES, 8 A.M., MARCH 16, 1936.

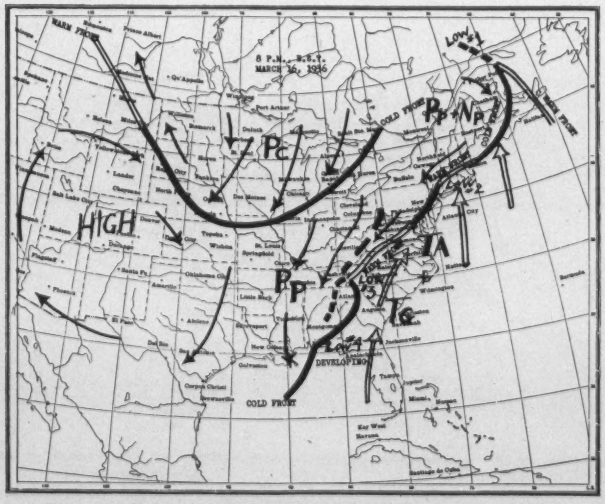

FIG. 7.—FRONTS, AIR WAVES AND AIR STREAMLINES, 8 P.M., MARCH 16, 1936.

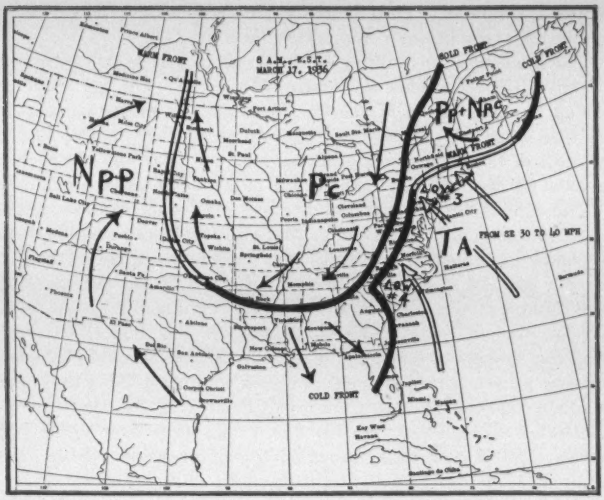

FIG. 8. — FRONTS, AIR WAVES AND AIR STREAMLINES, 8 A.M., MARCH 17, 1936.

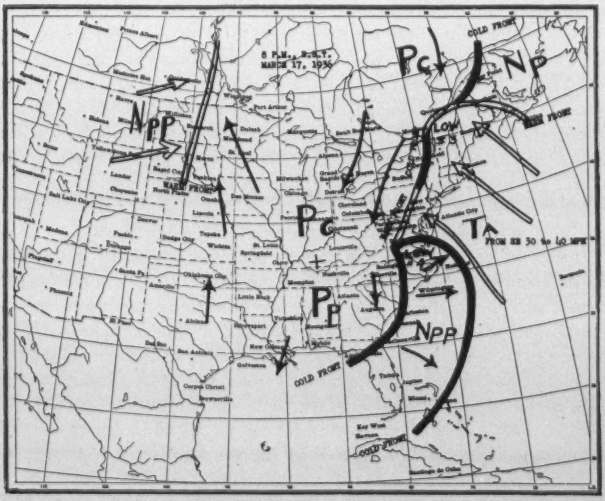

FIG. 9. — FRONTS, AIR WAVES AND AIR STREAMLINES, 8 P.M., MARCH 17, 1936.

conclude that rainfall as great as occurred in the Johnstown flood is not be to expected under conditions of appreciable thawing.

The importance of thawing in the floods of last March is illustrated in Figures 1–3 which show the approximate positions of the isotherms of mean air temperature on March 15, 16, and 17. It will be noted that for more than three days the average temperature was higher than 40 degrees over most of the watershed considered and that over the southern part, such as in the Shenandoah, temperatures of 50 to 60 degrees prevailed.

The series of low pressure centers causing the rains which accompanied this thaw form an interesting study. Maps for twelve-hour intervals during this period are presented in Figures 4–9. On the morning of March 15th, the rapid advance of warm air from the southwest over the eastern part of the country is noted, followed to the west by a cold front along which a new low pressure center (Low No. 2) is developing (Fig. 4). By evening of that day the new development has continued and moved as a wave into Tennessee while a third low pressure center is forming along the southern part of a second cold front over the Great Plains (Fig. 5). On the morning of the 16th it is noted that Low No. 2 is in the area of West Virginia, Ohio and Pennsylvania where it is causing fairly widespread rains as the tropical Gulf air slides up over the polar air to the north of the warm front (Fig. 6). At the same time, Low No. 3 has continued its development and is moving northeastward from Shreveport, La. It is this third system that causes most of the rain which occurs over the eastern mountain areas during the next 48 hours, and already on the evening map of the 16th, this system has become prominent as a rain producer (Fig. 7). Meanwhile, a fourth depression is forming along the cold front near Pensacola, Fla., and on the map of the 17th, it is noted that Low No. 4 is taking the place of Low No. 3 and the two together are producing heavy rains from South Carolina to Maine as the tropical Atlantic air moving from the southeast at 30 to 40 miles per hour at the surface is forced up over the cold air to the north and west (Fig. 8). By the evening of the 17th, Low No. 4 has reached its peak development and the overrunning of the tropical Atlantic air over the polar air to the west in the vicinity of the mountains is striking (Fig. 9).

It will be noted that in this case there is a strong outbreak of tropical Atlantic air flowing over the mountains, but rain in appreciable quantities occurred only along the fronts. The torrential rains associated with pure mountain lifting of unstable air are not present — that is, there is relatively little rain in the warm air mass itself. This is taken as an indication that the tropical air was stable. Temperatures along the coast support this conclusion and a meteorological airplane flight showed definitely that the air was stable and a greater amount of lifting would be required to make it unstable than could be afforded to it by the mountains.

DEVELOPMENT OF THE GROUND WATER SUPPLY OF THE CITY OF DAYTON, OHIO.

BY W. W. MOREHOUSE.*

[*Read September 22, 1936.*]

Dayton, Ohio, is a city of 220,000 population, and the suburban sanitary districts outside of the corporate limits, together with the adjacent city of Oakwood and a small community to the south, add another 30 000 to the population served by the water plant. It is a manufacturing center and essentially a city of homes, whose owners are proud of their lawns. Dayton is situated on the Miami River at the junction with two of its largest branches, the Stillwater and Mad rivers. These rivers cut their channel in ancient times through limestone beds and later the glaciers partially refilled these channels with glacial drifts of sand, gravel and till. It is in this glacial drift of the Mad River Valley that the ground water supply for Dayton has been developed. This river supplies more than half of the dry-weather flow of the Miami River. It is largely fed from springs, and the nearest sewage contamination is twenty-five miles away. The gravel deposits are coarser and allow a greater underground flow than in the other valleys, and from every point of view the Mad River Valley has proved itself the best source for the public water supply.

The history of Dayton water supply, like that of many other cities, is the story of a series of struggles to keep the supply up to the demand, and generally the demand remained ahead of the supply. The early history of the water supply is interesting in the variety of expedients that were attempted to find an adequate supply. The original water works plant was planned to be built at a location a half mile at least from the river and the present location, but after digging a large well it was discovered that very little water was available, and land was then purchased at the present location; a well 25 ft. in diameter and 15 ft. deep was constructed and the pumping station built, distribution lines laid and the system was put into service in March, 1870. It was soon discovered that the original well would not supply enough water, and a supply trench, or filtering gallery, was excavated from the well eastwardly about 100 ft. It was presumed that this was walled up and covered, although the records do not say.

The original well and supply trench could not furnish sufficient water, and a second supply trench, or long well, was constructed during 1871, at right angles to the original one. This was 108 ft. long, 23 ft. wide and 15 ft. deep. It was walled up with stone and a wooden roof built over it. This scheme did not come up to expectations, and by the close of 1872 the demand was again in excess of the supply and a new scheme was decided on and

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started. Three 20-in. cast-iron pipes were laid through the levee from an intake in the river to a valve chamber on the land side of the levee. This was connected to a filter gallery 8 ft. wide and 9 ft. high, constructed somewhat in the shape of a letter F, which surrounded the long well on two sides and one end. The gallery was walled up with stone, covered with large stone slabs and back-filled. The bottom of the gallery was at the elevation of the bottom of the well.

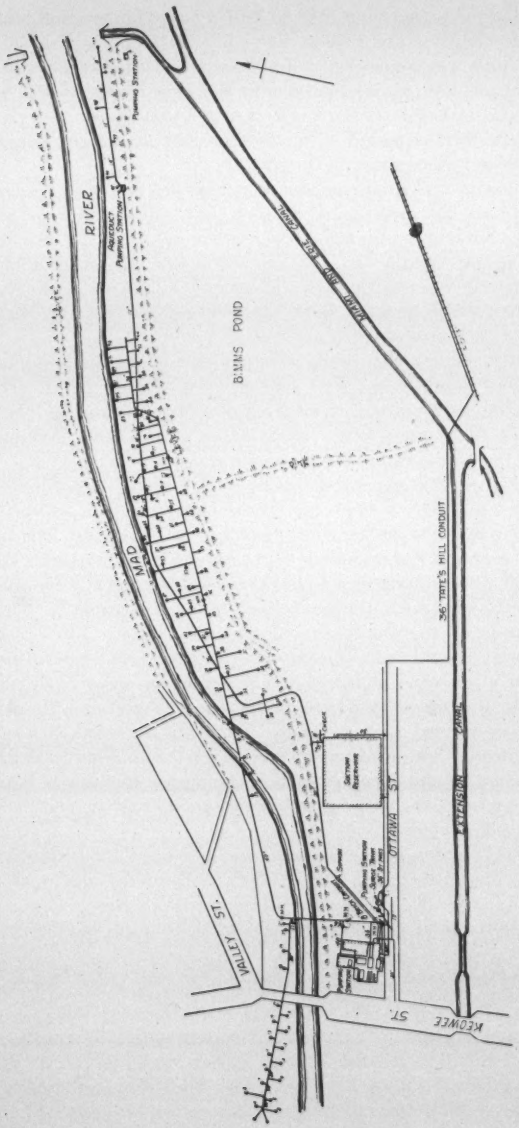
The idea was that river water filtering through the intervening gravel would be sufficiently purified and a sufficient supply would be available. This did not work out; the filter became clogged with river mud and sediment. The filter was cleaned, but no better results followed, and in the summer of 1874 raw rater was admitted directly to the long well by sluices cut from the gallery to the well. This was very unsatisfactory, and the following year a new well 18 ft. in diameter and 30 ft. deep was dug and walled up with stone. The expectation from this well was not realized as the lift was too great for the pumps to utilize all of the water, and in 1877 the original well was connected to the new well by a 24-in. pipe line and the pump suctions were placed in the new well.

During the summer of 1879, the demand again exceeded the supply and raw river water was again resorted to, which history relates "was very unsatisfactory to our citizens." During 1881 and 1882, another filter gallery was built. This extended from the new well easterly, then northeasterly. This was constructed 136 ft. long the first year and extended 164 ft. in 1876, making 300 ft. in all. The bottom was 26 ft. below the surface. The gallery was 4 ft. wide at the bottom and the walls of stone were 8 ft. high, and it was covered with large stone slabs spanning from one wall to the other.

During the summer of 1885, river water was admitted to the wells, causing much dissatisfaction to the water users, and finally arrangements were made with one Ezra Bimm who had an ice-house 1 800 ft. east of the water plant and who had several wells from which in winter he filled his ice ponds. The first water was brought to the plant in an open ditch, but the following year a 10-in. pipe was laid for this purpose.

During 1887, the first 8-in. wells were put down in the river bed. There were thirty of these of an average depth of 40 ft. They were connected to a 20-in. header connecting the wells to the 18-ft. well, and the water flowed by gravity. In 1890, five of these wells and ten new ones put in that year were connected to the suction of one of the pumps.

In 1891 and 1892, thirty new 8-in. wells were put down and connected by a 30-in. line to the end of the filter gallery. These wells produced a natural flow of 2 m.g.d.; a syphon at the end of the line increased this somewhat. Twelve more wells were connected to this 30-in. line in 1893 and 1894. In 1895, this line was connected directly to the suction of the main pumps.


RIVER
MAD
BIMMS POND
ABOVE DAM
Pumping Station
Pumping Station
Pumping Station

During the period from 1898 to 1901 a new 66-in. suction header was laid out to the river and a 60-in. line laid eastwardly, and the 30-in. line from the west was connected. All wells were tested and those producing 100 g.p.m. or over were connected to the new header system and the remainder abandoned. Several new ones were put down and connected. The system then consisted of ninety-four 8-in. wells, all connected by direct suction to the pumps in the station.

In 1908, a consulting engineer was employed to report on additional water supply. He investigated what is now known as the Tates Hill supply, but advised against this as the location was two miles up Mad River from the station and the cost of a connecting pipe line was prohibitive. A second engineer was employed the following year and his recommendation was the Tates Hill supply. Both engineers agreed that the Mad River was the logical future supply for the city.

In 1910, six new 8-in. wells were put east of the old group of wells, near the old Miami and Erie Canal crossing of Mad River. This was named the Aqueduct Group. This group of wells was connected to a 2 000-g.p.m. centrifugal pump with electric motor and discharged through the old 30-in. line to the end of the filter gallery.

The actual development of the Tates Hill supply began in 1911 and was finished in 1914. A 36-in. cast-iron conduit was built connecting the main station with the discharge of the pumps at Tates Hill. This line was 2.29 miles long and was connected to the suction of the pumps at the main station and was also arranged to discharge water into the 18-in. diameter suction well, and this was the manner in which it was operated. The water was dropped into the well and then was picked up again by the pumps, losing several feet of head. The reason for this was that because of suction piping two of the larger pumps could not get enough water to supply them in any other way. Twenty-six 8-in. wells were constructed in this development and connected to two 5-m.g.d. centrifugal pumps connected to electric motors. This development was designed to furnish 10 m.g.d., but the maximum amount furnished with two pumps running was 8.5 m.g.d. and for the most part the two pumps were run alternately, night and day, and furnished 5 m.g.d.

In 1921, a group of eleven wells was developed about one-half mile farther up Mad River, but it was poorly constructed and went out of service in a few years. This was called the Eastwood Group.

In 1925 the total supply for the city of Dayton was as follows:

Old Main Group	92 wells	12 m.g.d.
Aqueduct Group	6 wells	1 m.g.d.
Tates Hill Group	28 wells	8 m.g.d.
Eastwood Group	11 wells	5 m.g.d.
Total	137 wells	26 m.g.d.

The average daily pumpage for that year was 16.5 m.g.d. and the maximum was 20.5 m.g.d. This margin was entirely too close.

A well known firm of Consulting Engineers was employed to make a comprehensive study of the entire system. A survey was made and a very excellent report submitted with recommendations for the betterment of the entire system.

The land bordering on Mad River and extending from the lands at Tates Hill and Eastwood Park, to the Huffman Dam of the Miami Conservancy District, a distance of 2.5 miles and containing 600 acres, was purchased for future supply. An abandoned railroad right-of-way forms the northern boundary of this line, and the grade has been converted into a road, giving access to all that part north of the river.

An 84-in. precast concrete pipe conduit was constructed 2.5 miles long from the new well fields to the suction reservoir at the pumping station. This line was later extended with a 54-in. line of the same character for another mile up the river. These conduits were gravity lines and designed for a 50-ft. head. All specials, such as bends, reducers, manholes, etc., were cast in place. All pipe was manufactured in a casting plant near the west end of the work. Here the reinforcing cages were fabricated, the concrete poured and the pipe cured, first with steam under canvas covers, then stacked on end in the yard. Each pipe was numbered and the date of casting recorded, and no pipe was hauled to the job until its full curing time had elapsed.

The 84-in. pipe was hauled to the location on a standard flat car, on a standard gauge railway which had been built along the site of the conduit. The car was arranged with a special drop frame which allowed the pipe to ride low and facilitated loading and unloading. The smaller pipe was transported to the job on a 30-ton trailer, of the type used for transporting steam shovels or other heavy machinery. The excavation was made by a large drag line, and the pipe was placed in position by the same equipment. The pipe was laid upon foundation blocks which were poured to grade of the bottom of the pipe several days before laying pipe. The 84-in. line included one river crossing which was built as an inverted syphon, about 12 ft. below the regular grade line with vertical shafts at either end.

The report of the consulting engineers recommended developing the proposed additional water supply by constructing groups of 8-in. wells, served by a small pumping station. Actual practice deviated from this plan. Individual large wells were decided upon, each served by its individual vertical submerged pump. A contract was awarded to a water contractor for the furnishing of 10 m.g.d., and seven 38-in. diameter wells were constructed, in depths from 60 to 90 ft. These were of the type known as gravel wall and were constructed by what is known as the mud scow method. A 52-in. casing was sunk down to water level by hand. Then the mud scow or sand pump was used to do the digging. The mud scow was a large bailer similar to the ones used in drilling small wells, to bring up the

mud and water after drilling. The bailer used in these wells was 20 in. in outside diameter by 12 ft. long, weighing in the neighborhood of 2 tons. The lower end of the bailer was equipped with a flap valve which opened upward. A piston worked on the inside of the bailer and was attached to a piston rod somewhat longer than the shell of the bailer. The piston rod slid through a swivel guide attached to the top of the bailer and the hoist cable was attached to the upper end of the piston rod.

The procedure was this: The bailer was allowed to descend into the casing to the bottom, the piston dropped to the bottom of the bailer, the hoist was started pulling the piston upward in the bailer, which pulled the gravel and sand into the bailer through the flap valve in the bottom. When the bailer was full it was hoisted up and dumped and the process repeated. When the outside casing was bailed down to a depth of 30 ft. below ground level, the regular screen (38 in. in diameter in this case) was started down in the center of the outer casing. When the proper length of screen had been bailed down a blank casing was riveted to the top of the screen and bailed down until the well was at the proper depth. Screen and blank casing were added in lengths of 10 ft. for convenience. As the screen and blank inner casing were being bailed down, 1½-in. washed gravel was fed in between the outer and inner casing to form a screen or reservoir around the outside of the screen. The inner casing was cut off and encased in a concrete foundation, the top of which was set above flood level as a foundation for pump and shelter house. The pump was installed, the pump head and motor were set on the concrete foundation and the pump house was built. These were formerly of brick, for protection, but lately the houses have been built of galvanized sheets on a welded angle frame.

Since the construction of the first 15-m.g.d. additional supply, a survey of the water supply land has been made. A small drill rig with tools capable of drilling 200 ft. of 6-in. hole was purchased, and test wells were drilled 400 ft. apart over the larger part of the water supply land. Logs of each well were made, and each test well that had sufficient water bearing strata was pumped with a 6-in. centrifugal pump to determine the amount of water it would supply, and the probable amount a large well at that location would supply. This information is a matter of record which is incorporated in a report on file at the Water Department offices and available for future use.

It is the opinion of the writer that it is poor policy to install too large a pump in a new well. It is quite tempting to do this, but a pump of too great capacity will increase the velocity through the screen so much that it will pack the screen, or the surrounding gravel screen, with fine sand to such an extent as to cut the well capacity very considerably in a comparatively short time.

We have always used as coarse a screen as we can get, for the reason that the water-bearing strata are coarse material for the most part, and in the development of a new well, after it is finished, we pull into the well all

the loose sand that it is possible to pull from the surrounding gravel. This is done by a piston arrangement which fits the inside of the casing. By working this up and down, the loose sand which would later be brought into the well by the pump is pulled in beforehand and bailed out before setting the pump. As our water-bearing strata are very close to the surface we use a screen which is about 30 ft. shorter than the depth of the well, the first 30 ft. being sealed off to exclude surface water. Thus we have 30 ft. of natural sand-filter before any water can get into the well.

The Department has been fully equipped to construct its own gravel-wall wells for several years. Seven wells have been constructed totaling 20 m.g.d. in the past seven years. We use a derrick built of pipe, similar to that used for drilling oil, which can be erected or dismantled in half a day. A double-drum gasoline hoist is used for operating bailer and setting pumps, etc. For testing large wells, we have a 75-h.p. gasoline motor and a belt head for pumps. For testing 6-in. test wells we have a 6-in. self-priming, gasoline-driven, centrifugal pump on wheels.

Probably a few words in reference to cleaning our old 8-in. wells would be interesting. These 8-in. wells are connected to the suction header in a peculiar manner. The well is cut off about the height of the suction header, which is below water level. A special flange is attached to the well pipe to which is fastened a special tee. The lower end of the run is flanged to fit the flange on the well. The branch has a bell end into which the connection from the well to the header is leaded. This connection has a valve between the well and the suction header. The upper end of the run has an 8-in. thread and a 9-in. thread. In the 8-in. thread a special bronze plug is inserted to seal the well from surface water, and into the 9-in. thread a 9-in. pipe is screwed to carry the well above ground surface and to protect a 2-in. square steel bar which is attached to the bronze plug. The 2-in. bar is to operate the bronze plug when the well needs cleaning, or inspection. This equipment saves excavating 10 or 12 ft. below water level when cleaning a well.

To clean a well, the plug is removed, and the valve between the header and well is shut. First, live steam is blown into the well through a small pipe extending to the bottom of the well. After a short time a geyser of water, sand and mud is blown out of the well. This is repeated several times until the corrosion on the screen is loosened. Next, a centrifugal pump is attached to the top of the well by a flexible hose connection and the well is pumped vigorously. Frequently during the pumping, the pump is shut down, suddenly causing a surge of the water in the well, then the pump is started again. This procedure is repeated until the delivery shows decided improvement. In some cases, where we can make no impression on a well, the steam is applied at the top of the well, through a tapped cap. Steam pressure is applied, and the water in the well is blown through the screen out into the gravel strata. This generally clears the sand from the outside of the screen and improves the delivery.

The improvements made were not confined to water supply alone. A third 10-m.g. reservoir was constructed, several feeder lines were laid and a new pumping station built. This is entirely electrically operated, the power being purchased from the local power company. (See Figure 2.)

The system consists of the water supply as described and the electrical main pumping station. The distribution system is divided into two parts, the low service, serving the lower part of town and the high service, serving the hilltops. One of these high-service districts is served by a booster station from the low service, and we have a second small high service

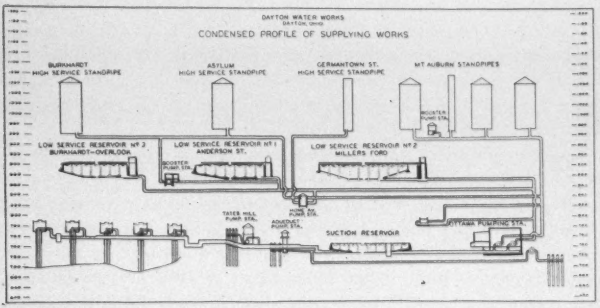


FIG. 2.— CONDENSED PROFILE OF SUPPLYING WORKS.

served by a booster from the high service. Three 10-m.g. reservoirs at the top of the low service give us 10-m.g. storage and balance supply for this service. Five 1-m.g. standpipes and one 90 000-gal. standpipe do the same for the high service.

We always aim to have sufficient capacity in wells to be able to furnish all water required with the largest group of wells out of service, with a well or two in reserve. During the lighter months we endeavor to operate each well at least once a week. Our wells run in capacities from 2 to 7.5 m.g.d.

The Dayton Water System at present is briefly as follows:

The supply comes entirely from wells, which are pumped by low-lift pumps discharging into an 84-in. conduit which brings the water to the suction reservoir at the station. The pumps in the main station send the water out to the distribution system at two pressures, 185-ft. head for the lower part of the city and 350-ft. head for the hilltop service. A booster station serves one high pressure area, pumping from the low-pressure lines. One small area, too high for the high service in summer, is served by a booster from the high service. At the top of the low service area are three 10-m.g. covered reservoirs which balance the demand on this service. On the high service four 900 000 gal. standpipes, one 1-m.g. standpipe and one 90 000 gal. standpipe act as storage and balance the high-service demand.

ADDITIONAL WATER SUPPLY FOR THE CITY OF NEW YORK.

BY WALTER E. SPEAR.*

[*Read September 22, 1936.*]

It is now twenty years since the Catskill water supply was first delivered to New York City. At that time the population of the city was about 5 400 000 and the consumption, including the supply furnished by private water companies, was 585 m.g.d. Now the population is estimated as about 7 364 000 and the total consumption, with the water delivered by private water companies and the draft on the city's works by other municipalities, is not far from 1 000 m.g.d.

In 1921, the Board of Water Supply was requested by the Board of Estimate and Apportionment to seek new sources of water supply. The average daily use of water in the city, plus the draft by outside municipalities was then 743.1 m.g., and at the annual rate of increase at that time of about 30 m.g.d. it appeared that a new water supply would be required by 1935. But for the financial depression, which was not reflected in New York City's water consumption until 1931, it is probable that an additional water supply might have been needed earlier because the total average consumption in New York City and the amount furnished other municipalities in 1930 was somewhat larger than at present, 1 007 m.g.d., that is but 32 m.g. less than the estimated safe capacity of the existing works, municipal and private, estimated at 1 039 m.g.d.

The effect of the depression in reducing the domestic and industrial use of water, and the work of the Department of Water Supply, Gas and Electricity in reducing domestic waste and underground leakage, cut down the consumption in 1931 by about 4.5 per cent. The use of water fell off slightly until 1934 when it increased to 978 m.g.d., but it decreased again in 1935 to 970 m.g.d. For the first 8 months of 1936 the consumption is reported to have averaged about 28 m.g.d. more than for the corresponding months of 1935, promising a rate of about 1 000 m.g.d. for the year 1936.

The continued sub-normal consumption and generally abundant rainfall since 1931 have concealed the serious water supply situation that now exists in New York. It has been urged in some quarters that the city has definitely ceased to grow and will need no more water if adequate measures are adopted to reduce the per capita consumption, but there is no justification for this argument. It has come to be realized that the present margin of safe yield from existing sources of about 5 per cent. over the present consumption is unsafe, and the administration has decided this

*Chief Engineer, New York Board of Water Supply.

year to proceed with the development of an additional supply from the Delaware project.

The investigations of the Board of Water Supply beginning in 1922 showed that any large additional supply of water could be most advantageously taken from the watershed of the Delaware River; but that river is an interstate stream, and it was realized that the consent of New Jersey and Pennsylvania might have to be obtained even for the development of the tributaries of the Delaware River within the state of New York. Accordingly, the city attempted to negotiate treaties with those states for a division of the Delaware River waters, and treaties for that purpose were signed by representatives of New York, New Jersey and Pennsylvania in 1925 and 1927, but both treaties failed of legislative ratification, except in New York.

After taking competent legal advice, the city next proceeded to attempt the diversion of the waters of the upper Delaware River watershed without further effort to secure the consent of New Jersey and Pennsylvania. Upon the recommendation of the Board of Water Supply, the Board of Estimate and Apportionment on January 12, 1928, approved the development of a supply of 100 m.g.d. from the Rondout creek of the Hudson River watershed and of 570 to 600 m.g.d. from the Delaware River watershed. Approval of the project was given by the State Water Power and Control Commission on May 25, 1929. The State of New Jersey had brought suit before the United States Supreme Court on May 13, 1929 to enjoin the City and the state of New York from diverting any water from the Delaware River watershed. On May 4, 1931, the Court approved the recommendations of the Special Master who heard the case and handed down a decision granting to New York City the right under certain conditions to take 440 m.g.d. of water from the Delaware River watershed. In the meantime, after the filing of the Special Master's report, an authorization of $50 000 000 for the Board of Water Supply was made in April, 1931 to construct the southerly section of the Delaware aqueduct between Kensico and Hill View Reservoirs, which was common to all projects for an additional supply; but, when contracts for that section were about ready to let and real-estate maps had been filed for other portions of the Delaware aqueduct, all new construction was suspended late in 1931 by reason of the city's financial condition. Most of the authorization for the construction of the Kensico-Hill View Tunnel was shortly after suspended, and the personnel of the Board of Water Supply was reduced to the minimum required to complete the work then in hand on the City Tunnel No. 2 and to carry on some preliminary surveys and office studies on the Delaware project.

With the decision, this year, to proceed with the Delaware project, the full authorization of funds to construct the reservoirs and aqueducts of the first stage of construction was not considered practicable, and in the Capital Outlay Budget of 1936 the sum of $20 000 000 was included to

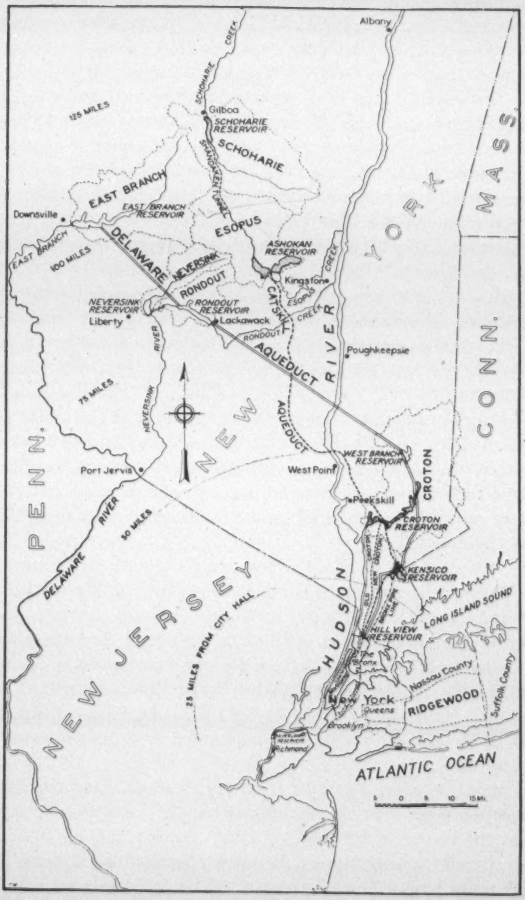


FIG 1.— NEW YORK CITY'S WATER SUPPLY, SHOWING THE CATSKILL, CROTON, RIDGEWOOD AND DELAWARE SYSTEMS.

acquire the necessary real estate, to construct the shafts of the Delaware aqueduct from the Rondout Reservoir to Hill View Reservoir near the city and to do some preliminary work on the first dam to be constructed on Rondout Creek. The sum of $17 500 000 was subsequently authorized for these purposes and the necessary expenses of the Board during the first year.

The real estate maps have since been approved and signed by the Board of Estimate and Apportionment and the necessary steps have been taken to secure the appointment of the Commissioners of Appraisal and to take title to the lands and obtain the necessary easements. Contracts for the shafts are in preparation and it is expected that some of these contracts will be advertised late in the fall of 1936.

As shown in Figure 1, the Delaware project as now constituted comprises the development of the Rondout Creek, a tributary of the Hudson River, and of the Neversink River and the East Branch of the Delaware River, and the construction of an aqueduct to the West Branch Reservoir of the Croton watershed to pick up high-level Croton water and thence to Kensico Reservoir of the Catskill system and to the Hill View Reservoir of that system just over the city line in Yonkers. The Rondout Creek is estimated to yield 100 m.g.d. and the two tributaries of the Delaware River 440 m.g.d. as permitted by the decision of the U. S. Supreme Court. Other tributaries of the Delaware River included in the original Delaware plan, the Willowemoc Creek, the Beaver Kill and the Little Delaware River, which were estimated to yield 160 m.g.d., are excluded by the terms of the Supreme Court decision.

The valleys of the Rondout Creek, the Neversink River and the East Branch of the Delaware River contain deep deposits of glacial drift, and the explorations on the proposed dam sites indicate that the depth and character of these deposits are such as to require in each case that a tight masonry core wall be carried to rock by the caisson method adopted at Bingham, Maine, by the Central Maine Power Company and at the main dam and dike of the Quabbin Reservoir that is being constructed on the Swift River by the Metropolitan District Water Supply Commission of Massachusetts.

The main Delaware aqueduct from the Rondout Reservoir to the Hill View Reservoir near the city, 85 miles in length, comprises three pressure tunnels in the bed rock, the Rondout-West Branch tunnel, 46 miles long, the West Branch-Kensico tunnel, 24 miles long, and the Kensico-Hill View tunnel 15 miles long. The all-pressure tunnel type was adopted for the first two sections of the aqueduct because the elevations of the Rondout and West Branch reservoirs are so high that the gradients are generally above the ground surface and no other type of construction is practicable. The pressure tunnel type of construction is also to be used between the Kensico and Hill View reservoirs for the reason that Westchester county has developed so much during the years since the Catskill aqueduct was

constructed in the same locality that where even topographically possible an aqueduct of the cut-and-cover type, largely adopted for the Catskill aqueduct, would involve prohibitive real estate costs and local opposition.

The aqueduct from the Neversink Reservoir to the Rondout Creek will be constructed as a short grade tunnel, about 3 miles in length, that from the East Branch Reservoir to the Rondout Reservoir a grade tunnel, about 25 miles in length.

The estimated cost of the first stage of the Delaware project, comprising the Rondout and Neversink reservoirs and the aqueduct to the city is $210 000 000, and it will take at least 7 or 8 years to deliver the first water to the city; that of the second stage, comprising the East Branch Reservoir and the aqueduct to the Rondout Reservoir, $62 000 000; and the total cost of the entire project as now constituted $272 000 000. It is planned to begin construction of the second stage at such time that the additional water from the East Branch of the Delaware River can be delivered when needed.

The development of the Delaware project is under the direction of the Board of Water Supply comprising three Commissioners, George J. Gillespie, President, Henry Hesterberg and Rufus E. McGahen.

PROCEEDINGS.

March 1937 Meeting.

Hotel Statler, Boston, Mass.

Thursday, March 18, 1937.

President Harry U. Fuller in the Chair.

The President announced the death of James W. Killam and Albert D. Flynn.

Arthur C. King reported for the Committee on Legislation for Massachusetts.

A paper "Silvicultural Aids in the Control of the Gypsy Moth in Watershed Forests," illustrated, was read by A. C. Cline, Assistant Director, Harvard Forest, Petersham, Massachusetts.

The President announced elections to membership as follows:

Members: George G. Bogren, Member, Water Board, Wayland, Mass.; Kenneth W. Robie, Civil Engineer, Brookline Water Department, Brookline, Mass.; Hugh J. McLaughlin, Water Commissioner, Chelsea, Mass.

Corporate Member: Water and Sewer Commissioners, Westboro, Mass.

A paper "Transite Pipe River Crossing at Scituate, Mass.," illustrated, was read by Frank A. Marston of Metcalf & Eddy, Boston, Mass.

A paper "Transite for Well Pipe and Screens," illustrated, was read by William J. Lumbert, Superintendent, Water Department, Scituate, Mass.

A paper "Experience with Transite Pipe," was read by A. B. Rich, of Fay, Spofford & Thorndike, Boston, Mass.

A paper "Our Introduction to Transite Pipe," illustrated, was read by Harold L. Brigham, Superintendent, Water & Sewage Commission, Marlborough, Mass.

A paper "Underwriters Tests of Transite Pipe," was read by Charles W. Sherman of Metcalf & Eddy, Boston, Mass.

The above papers were discussed by Percy Charnock, Theodore L. Bristol, and Robert W. Mawney.

April 1937 Meeting.

Hotel Burritt, New Britain, Conn.

Thursday, April 29, 1937.

President Harry U. Fuller in the Chair.

The President presented the following guests: Messrs. Goodwin, Chairman of the Hartford Metropolitan District; Humphrey, Chairman of the New Britain Water Board, and the Honorable George A. Quigley, Mayor of the city of New Britain.

William W. Brush discussed the decision of the United States Supreme Court relative to taxation of income derived as an employee of a municipal water works system.

Secretary Gifford announced the election by the Executive Committee of the following to membership:

Members: William Edward Hogan, Water Board, Somerville, Mass.; J. R. Dangler, Cast Iron Pipe Institute, Washington, D. C.

Associate: National Tube Company, Boston, Mass. (reinstated).

A paper "New Type of Concrete Standpipe," was read by J. W. Holden, Engineer, New Britain Water Board.

A Round Table Discussion on the subject, "Procedures for Making and Financing Water Extensions," was opened by Caleb M. Saville, Manager, Water Bureau, Metropolitan District, Hartford, Connecticut, and Dwight Hall, Superintendent, Bridgeport Hydraulic Company, Bridgeport, Connecticut. Messrs. Henry T. Gidley, Francis H. Kingsbury, Patrick Gear, Irving Rudd, George F. Merrill, Theodore L. Bristol, David E. Moulton, and Frank S. Brainerd took part in the discussion.

Papers on "Sizes of Meters for Services," were read by Roger W. Esty, Superintendent, Water Board, Danvers, Mass., and Julius M. Carley, Water Department, Southington, Conn.

May 1937 Meeting.

Hotel Weldon, Greenfield, Mass.

Thursday, May 20, 1937.

President Harry U. Fuller in the Chair.

The President introduced Mr. Wesley B. Foss, Secretary of the Greenfield Chamber of Commerce.

The Secretary announced the election to membership of: Carter H. Lamb, Valuation Engineer, Jamaica Water Supply Company, Jamaica, N. Y.

A paper "Thread Gauge Design and Discussion of Threading Problems," was read by Oscar E. Kohler, Chief Engineer, Greenfield Tap and Die Corporation.

A paper "Brattleboro, Vermont, Water Works," was read by George E. Hawkins, Superintendent, Brattleboro Water Department.

Mr. W. Guy Classon, Superintendent, Leominster Water Department, showed motion pictures taken on a recent trip to the west coast, and also of the construction of a rapid sand-filtration plant and sewage-disposal works, recently completed at Leominster.

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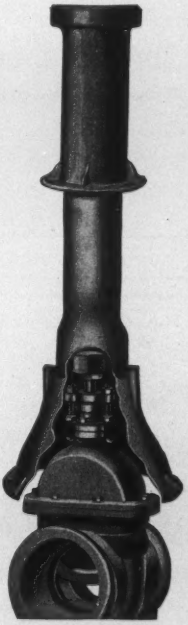

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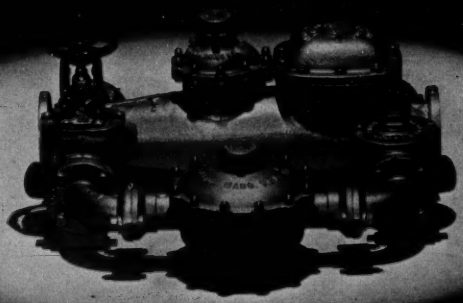

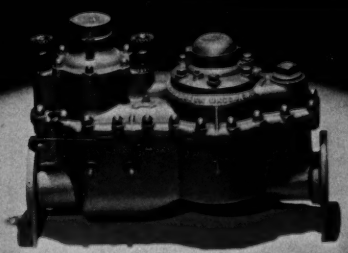

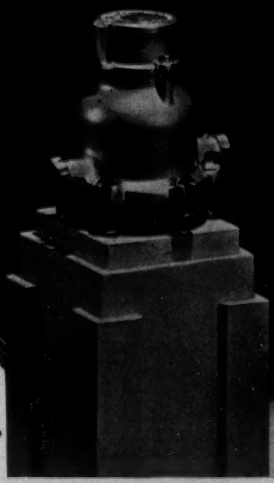

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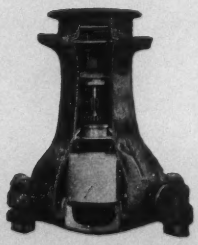
While small boys have grown to men . . . this Company has consistently been the leader in every worthwhile improvement in the design and construction of all types of water meters.

And how Water Works men "follow the leader" . . . over six million Trident and Lambert Water Meters made and sold the world over!

Neptune Meter Company (Thomson Meter Corp.), 50 West 50th St. (Rockefeller Center), New York City . . . also . . . Neptune Meters, Ltd., 345 Sorauren Ave., Toronto, Canada . . . A meter for every type of service.



Follow this lead . . . see these modern interchangeable parts inserted into the casing of a typical Trident Water Meter that was a leader back in 1899 . . . Trident and Lambert Water Meters are now and always have been leaders in the principle of interchangeability.

TRIDENT
and LAMBERT
WATER METERS

IN BRISTOL VIRGINIA

"THE TWIN CITY"

TENNESSEE

Nestled in a valley between the Cumberland and Blue Ridge Mountains and located squarely on the State Line between Virginia and Tennessee, Bristol, although one community, is in reality a two-state city. While its civic organizations have but one body, it has two mayors, two city councils, two chiefs of police—in fact, a "pair" of all political officials. Elevated 1,700 feet above sea level and having an average temperature of 61 degrees, Bristol's slogan is "A Good Place To Live."

Bristol is a progressive industrial city. One of the largest overall factories in the world is located here. Other industries produce lumber products, paper, hosiery, women's clothing, trailers and mine cars. In the beautiful surrounding country the chief agricultural products are tobacco, corn, wheat, dairy, poultry and produce from general farming.

Both City Water Departments are users of Pittsburgh Water Meters, Bristol, Tennessee, having purchased 3,000 in 1936.

PITTSBURGH EQUITABLE METER CO.

MERCO NORDSTROM VALVE CO.

MAIN OFFICES PITTSBURGH, PA.

NEW YORK	LOS ANGELES	TULSA	HOUSTON	MEMPHIS	BUFFALO
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THE STANDARD FOR OVER 50 YEARS

Automatic Pressure Control Valves
Water Works Specialties

Altitude — Pressure Reducing — Surge
Relief and Combination Valves
Portable Fire Hydrants
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ROSS VALVE MFG. CO., INC. TROY, N.Y.

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Water delivered through dirty pipes
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Incrusted water pipes mean inefficiency
and loss of Fire Protection.

We Guarantee the Results of Our
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National Water Main Cleaning Co.

50 Church Street New York City

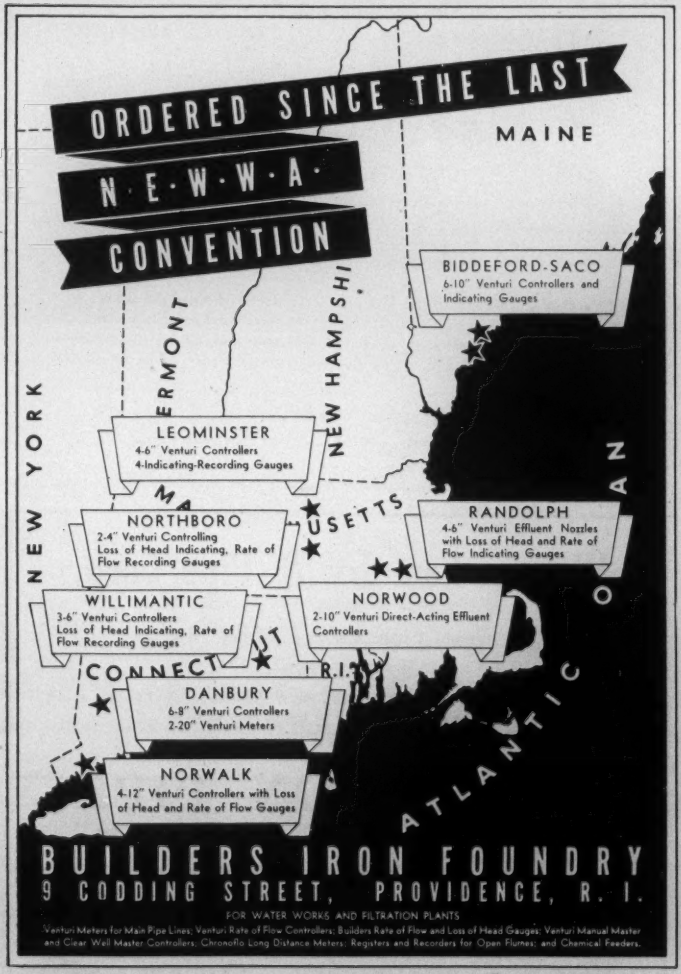
Water-works Practice and Theory

WHEN you wish information on any phase of water-works practice, let us look up the subject for you in our Journal.

In one of the issues there will be the material you want.

Back numbers of the Journal are available at **$1.25** per copy.

New England Water Works Association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FOR WATER WORKS AND FILTRATION PLANTS
Venturi Meters for Main Pipe Lines; Venturi Rate of Flow Controllers; Builders Rate of Flow and Loss of Head Gauges; Venturi Manual Master and Clear Well Master Controllers; Chronoflo Long Distance Meters; Registers and Recorders for Open Flumes; and Chemical Feeders.

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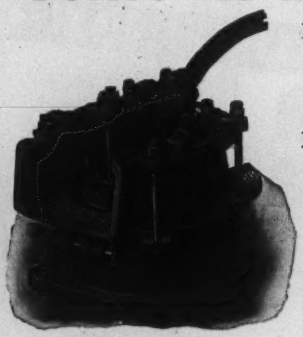
PUMPS and ACCESSORIES



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WORKS: PHILLIPSBURG, N. J. and EVERETT, MASS.

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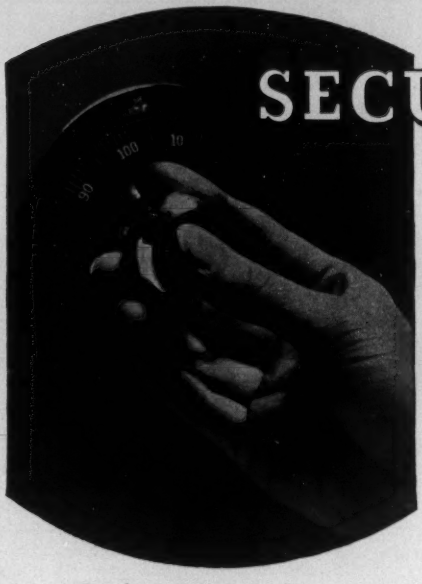
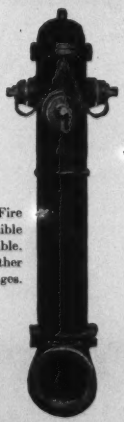


SECURITY

You can match the security of the bank vault with Darling security of water control.

★



All parts of Darling Fire Hydrants are accessible and interchangeable. There are many other important advantages.

We don't know who said "Security maketh a goodly warm feeling"—but we know he was right. It's a great thing to know that the control elements of your water system are giving safe, dependable service. And that's what you get from Darling Valves and Fire Hydrants.

It is also good to know that these safe and dependable elements are constantly saving money and worry through low maintenance costs. No wonder the users of Darling Products are pleased with them!

The nearest Darling representative may already have the solution to your particular valve or hydrant problems. We'll be glad to consult with you!

DARLING VALVE & MFG. CO.

Williamsport, Pa.

Representatives in

New York Philadelphia
Oklahoma City Houston

Darling Valves are the choice of those who insist on performance.

DARLING

GATE VALVES *and* FIRE HYDRANTS

"Never a Failure in Over 22 Years!"

WE like to talk about saving in material and labor—the strong, tight, flexible joints made with Hydro-Tite—the thousands of miles of pipe jointed with it.

More important, perhaps, is the fact that with hundreds of users—each laying pipe under different conditions—not a single one has reported a Hydro-Tite failure.

We make it very easy for you to change over to Hydro-Tite. Our Data Book outlines a novel proposition. Write for it.

A Symbol of Quality

A Dependable Self-Caulking Joint Compound

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HYDRAULIC DEVELOPMENT CORPORATION

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Here's the INSIDE STORY on Low Pumping Costs!

Constant C-140 is responsible for Permanent Savings on this Non-Tuberculating Pipe . .

ISN'T THIS TRUE? Since water pipe is laid solely for the purpose of carrying water, in the last analysis your investment is not in pipe but in water transportation facilities.

Why, then, in figuring the cost of a new main, should the yardstick be cost per foot of pipe? Isn't it much more logical to evaluate a water main primarily on its effectiveness as a water carrier?

And not solely on its effectiveness when new . . . all too often water mains are at the mercy of tuberculation, which constantly decreases the delivery capacity of ordinary pipe.

Hence, can you afford to pass up the savings in pumping costs offered by Transite Pipe, as already proved by hundreds of installations?

C-140 is the conservative minimum coefficient of Transite . . . not merely when first laid . . . not merely after five years or ten . . . but indefinitely.

1500' OF TRANSITE PIPE from a city main into an industrial plant were tested by The Pitometer Co., N. Y. C. Conclusion —"From a study of the data, we believe that an average coefficient of C-145 is very nearly correct for this pipe."

Made of asbestos and cement . . . and hence non-metallic . . . Transite Pipe cannot tuberculate. From the day it comes off the polished steel mandrel that forms its smooth interior wall, this pipe's all-important value of C stays unchanged. And thus pumping costs remain permanently low.

Send for a copy of our brochure, containing the complete story of Transite's important installation and maintenance savings as well as low pumping costs. Write to Johns-Manville, 22 East 40th Street, New York City.

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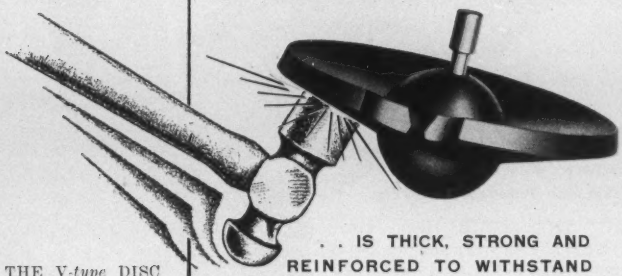
TRANSITE PRESSURE PIPE

An Asbestos Product

L. H. NASH

A NATIONAL NAME FOR ACCURATE DEPENDABLE ECONOMICAL WATER METERING

THE NASH *V-type* DISC



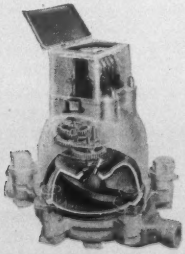
. . IS THICK, STRONG AND REINFORCED TO WITHSTAND MORE THAN 700 BLOWS A MINUTE

THE *V-type* DISC

Stronger for longer life

Larger for fewer motions and less wear

And with a measuring motion as free from friction as the nutation of a top



The L. H. Nash Water Meter

The losses in water revenue and the expense of time-consuming replacements of measuring discs that snap either at the first overload, or during periods of sustained operation at capacity, are seldom found in the service records of L. H. Nash Water Meters.

For the V-Type Disc of this rugged meter is thicker and stronger . . . yet is reinforced, too, as an added factor of safety to successfully withstand the rapid hammer-like blows caused by rapid nutation at full flow . . . blows that occur in disc meters despite every refinement in design.

And the stronger disc of the L. H. Nash Meter is only one of the many advanced features which result in continued accuracy of measurement . . . with the least trouble . . . at the lowest overall cost.

Why not write for complete information?

NATIONAL METER COMPANY

4203 FIRST AVENUE : BROOKLYN, N. Y.

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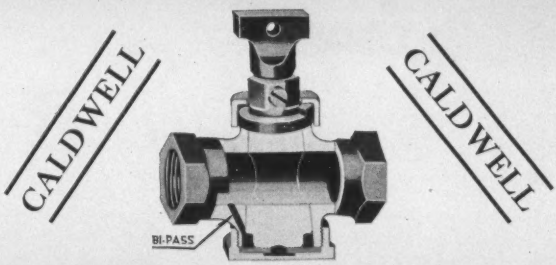
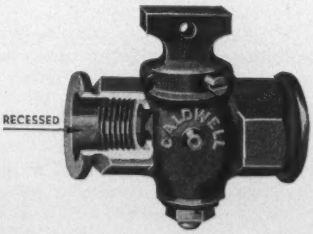


Our INVERTED KEY cocks have EXTRA HEAVY bodies SOLID PLUGS and FULL SIZED Round Water-way. Notice that the BI-PASS is so constructed that No Sand, Grit or Sediment can come in contact with the bearing surface of the plug.

These Cocks furnished with Non Breakable, Non Corrosive Manganese Bronze Tee Heads.

Guaranteed to turn easy under all pressures and conditions.



EXTRA HEAVY RECESSED NEWPORT CURB COCKS

Recessing permits the threads on the service pipe to be entirely covered. This retards CORROSIVE action and gives greater strength at the weakest point on the service.

"Boxfindr"

GEO. A. CALDWELL CO.

Mattapan Square

BOSTON, MASS.

"Pipefindr"

Crash!!

Fire!

Fire hydrants must contend with TWO kinds of emergencies

TODAY'S traffic conditions have brought a *new* challenge to fire hydrant design. In addition to fire service requirements, hydrants must now be strong enough to withstand glancing blows from automobiles and trucks, and if broken by a smashing impact, their design should permit speedy and economical repair.

Kennedy SAFETOP Fire Hydrants meet these new conditions squarely. Their exceptionally large standpipes will withstand any ordinary blow. However, if struck by an impact which *no* hydrant could resist, all damage to the hydrant is limited to the inexpensive Safety Breakable Section of the standpipe and stem.

Your community needs the double protection assured by Kennedy SAFETOPS—the fire hydrant that meets both the fire service and accident emergency requirements of TODAY.

The Kennedy Valve Mfg. Co., Elmira, N.Y.

KENNEDY
SAFETOP FIRE HYDRANT

REG. U.S. PAT. OFF.

EDDY

Valves and Hydrants

Distinctive Features of the EDDY Valves

ONLY three parts are moved by the Stem—the ball and the two gates. Each gate is hung from one of the trunnions on the ball. A convex surface at the base of each trunnion fits a concave surface on the back of the gate. This allows the gates to adjust themselves properly to their seats. . . . The gates, being free to revolve on the trunnions, do not always seat in the same position. Gates are center bearing and adjustable. They are forced to their seats with equal pressure at all points. There are two hooks on the ball on sizes 4-inch and larger, which loosely engage with the gates.

Distinctive Features of Our New Swivel Top Hydrants

THIS new hydrant has all the advantages of the popular EDDY fire hydrant, plus several additional features. . . . Nozzles are in a short flanged section of the standpipe, which may be unbolted and turned to different positions. . . . Should the standpipe be broken, only the casting below the swivel head need be replaced. . . . To raise the hydrant to conform to a new grading, simply insert a flanged extension piece below the swivelhead. No digging. To add a steamer nozzle at any time, it is only necessary to replace the swivel head with one having a steamer nozzle.

Besides valves and hydrants to meet the most exacting demands of waterworks service, the complete EDDY Line includes: valves for sewage disposal works; valves for steam, gas and oil; check valves, foot valves, plug valves, shear gates, indicator posts, etc. EDDY valves are made in three classes: Iron Body Bronze Mounted, All Bronze, and All Iron Valves.

Eddy Valve Co.

WATERFORD NEW YORK

LEAD PIPE

Used by leading WATER WORKS for over 50 years. If proper weight is used, combined with good workmanship, your FINAL COST is assured, as REPAIRS are unnecessary. LEAD PIPE is sufficiently ductile to conform to any sagging condition. There is no RUSTING of pipe or joints; no DISCOLORATION of water.

LEAD WOOL

Every Atom Pure Lead

For calking pipe joints under the most difficult conditions; for overhead joints or in wet places where the use of molten lead is not only impracticable but dangerous — LEAD WOOL may be used to advantage.

It makes an absolutely tight joint which will withstand the highest pressure, yet be sufficiently elastic to allow considerable sagging or settling of the pipe, without danger of leaks.

Compared with the poured joint, in this respect, the superiority of LEAD WOOL is apparent.

LEAD PIPE TIN-LINED LEAD PIPE
PURE BLOCK TIN PIPE SOLDER
DUTCH BOY WHITE LEAD & RED LEAD

Highest Quality Soft Virgin Omaha Pig Lead

NATIONAL-BOSTON LEAD CO.

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FOR CAST-IRON PIPE
FOR COLD WATER METERS

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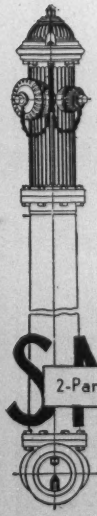


Minimum Trouble and Expense When Breaks Come

THE Smith Two-Part Standpipe Hydrant was designed to make the expense as small as possible when there is a break as the result, for instance, of careless auto truck driving.

The upper section will always break first. It can easily be renewed. And of course there can be no leakage of water because the main valve is of the compression type.

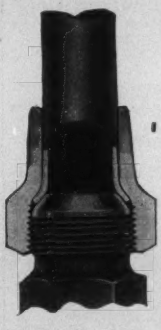
SMITH
2-Part Standpipe Hydrant

THE A. P. SMITH MFG. COMPANY
EAST ORANGE, N. J.

DOUBLE-LID METER BOX COVERS

Ford double-lid meter box covers provide dead-air insulation in neck of cover and conserve heat in the meter box. Top lids are provided with the Ford Worm Lock. Write for catalog of meter box covers, double and single lid.

The Ford Meter Box Co.
WABASH, INDIANA

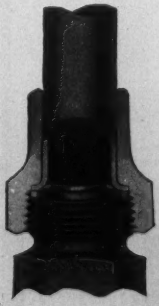


HAYS RADIUS SEAL JOINT

This type of joint was designed to meet the specific requirements of certain Water Departments and is available directly on all styles of Hays Copper Service fittings.

HAYS DOUBLE SEAL JOINT

This joint features a 2-faced pipe flare (45° + 90°) and a 2-faced machined seal (45° + 90°) which is by test the strongest flanged connection available. Available by Adapter on any Corporation Stop and directly on more than 400 styles and sizes of Hays Service and Plumbing fittings.



HAYS begins at the MAIN

Since 1869 Hays has built to the exacting requirements of the Water Works Industry. Tap the Main with a Hays Tapping Machine and follow through with dependable Hays Service Fittings for iron pipe, lead or copper. Hays Corporation Stops and Curb Stops are made in all styles and sizes. Write for Catalog.

USE THE HAYS COPPER SERVICE METHOD

Hays Adapts Copper to meet any Requirements

As a pioneer in the development of copper pipe services, Hays has popularized its use by making it easily adaptable to meet existing line equipment economically. Every community can have the advantage that copper pipe makes possible from Main to Curb to House with the Hays Copper Service Method. Let us have your problem.

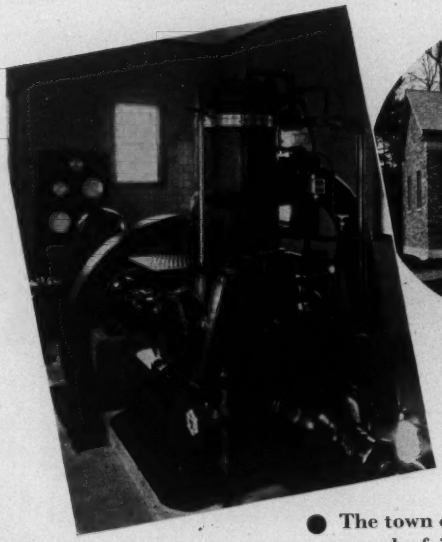
HAYS MFG. CO.

ERIE, PA.

HAYS

PLUMBING BRASS
BATH FIXTURES
COPPER PLUMBING
GAS SERVICE GOODS
WATER SERVICE GOODS
IRON PRODUCTS
TAPPING MACHINES

WATER PROBLEM SOLVED BY NEEDHAM, MASS. • • • •



X. Henry Goodnough, Inc.
Engineers

● The town of Needham can well be proud of its new water pumping station, for as city officials state, "Our water troubles will be over for a good many years to come." Believed to be the first Municipal Diesel driven Turbine Pump installation in New England, this system is equipped with a Fairbanks-Morse single-cylinder Diesel which powers an F-M Turbine Pump to economically supply the best quality water from an underground reservoir.

Needham, as well as many other New England municipalities equipped by Fairbanks-Morse, is showing a substantial saving in operating costs. Get the facts on F-M power and pumping equipment by writing to Fairbanks, Morse & Co., 178 Atlantic Avenue, Boston, Mass.

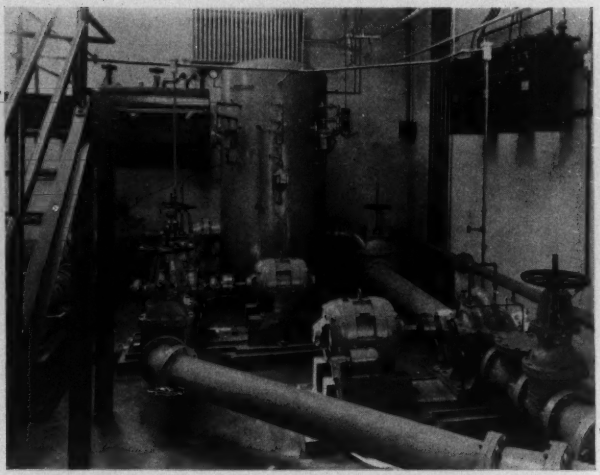
FAIRBANKS-MORSE & CO.

POWER • PUMPING • AND • WEIGHING • EQUIPMENT

DE LAVAL

Centrifugal Water Works Pumps

With Continuous Automatic Vacuum Priming



Low-Lift Pumps in the Ellis Water Treatment Plant
at Norwood, Massachusetts

Illustration shows two motor driven De Laval pumps with combined vacuum and sand chambers in background. Each pump has a capacity of 2 MGD against a total head of 41 feet including over 16 feet suction lift. Vacuum pumps can be seen back of stairway under platform.

This is another installation where our vacuum priming system successfully handles a large quantity of air continuously from a field of driven wells.

The plant was designed by X. Henry Goodnough, Inc., Boston, Mass., and our contract included all pumping equipment and mixing machinery delivered and erected.

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OF NEW ENGLAND

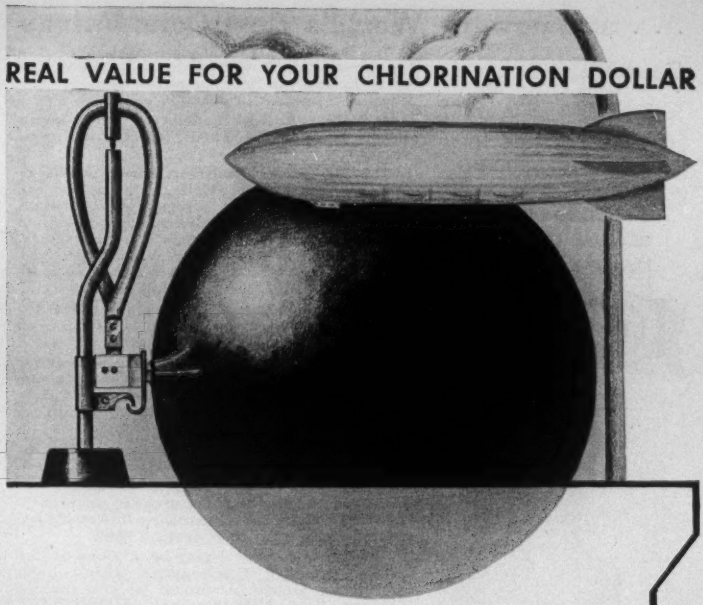
80 Federal Street Phone LIB erty 5993-5994 Boston, Mass.

New England Representative for

DE LAVAL STEAM TURBINE COMPANY

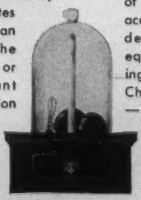
TRENTON, N. J.

REAL VALUE FOR YOUR CHLORINATION DOLLAR



Chlorination requires FLOATING POWER, TOO

POWER that floats on the water inside a bell jar controls the operation of a W&T Visible Vacuum Chlorinator No hidden diaphragms, no springs—only a needle valve operated by a floating hard rubber ball regulates the inflow of chlorine maintaining an even, constant pressure whether the equipment is feeding at its maximum or minimum. And not only a constant regulation, but a constant protection against escape of gas should the operating water supply fail. Immediately, the floating ball drops, seating the needle valve, and the gas is cut off until the trouble may be investigated and operation resumed.



Such carefully thought out design, coupled with meticulous attention to every small detail of material selection and manufacturing, accounts for the recognized accuracy, dependability and long life of W&T equipment. When specifying or purchasing equipment for the important job of Chlorination, be sure you call for W&T — for over twenty years the accepted standard of Real Value in the Chlorinator field.

Ask for Technical Publications 38, 157 and 158.

"The Only Safe Water is a Sterilized Water"

WALLACE & TIERNAN CO. INC.

Manufacturers of Chlorine and Ammonia Control Apparatus

NEWARK, N. J

Branches in Principal Cities

CHLORINATORS FOR WATER WORKS • SEWAGE PLANTS • SWIMMING POOLS • INDUSTRIAL PLANTS

SA-71

. . . and now the World's Best Cone Valves

Cone Valve Is Available for—

ALTITUDE CONTROL
PRESSURE REDUCING
CHECK VALVE
SLUDGE LINES
SEWAGE DISPOSAL
MANUAL CONTROL

WORLD'S BEST AUTOMATIC Valves

For more than 30 years Golden-Anderson has been known as the manufacturer of the WORLD'S BEST AUTOMATIC valves.

Now, Golden-Anderson offers you the supreme development in the Thru-Flow or Cone type Valves.

In the new Golden-Anderson Thru-Flow Valve the plug is unseated axially, rotated, and reseated axially, all in one free and frictionless operating movement and drop tight in either open or closed position; this is accomplished by means of a simple and sturdy patented toggle type rotor mechanism which is encased in a bath of oil in the operating head.

The valve itself is just as simple and sturdy, the full flow plug being stronger than pipe of the same area.

And each unit — plug, valve body, operating head, and automatic control — is separate and distinct, designed so that any one unit may be removed from the line without disturbing other units.

The valve stem may be repacked under pressure. The operating head cylinder stem may be repacked without draining the oil bath.

There are no balance ports, grooves, or cams in the valve. There are no cams, no line contacts, no stem screws in the operating head.

The automatic control may be any one of the several patented mechanisms which have established Golden-Anderson as the manufacturer of the "World's Best" AUTOMATIC Valves.

If you want increased efficiency and unfailing automatic control, ask the Golden-Anderson Engineers for further details.

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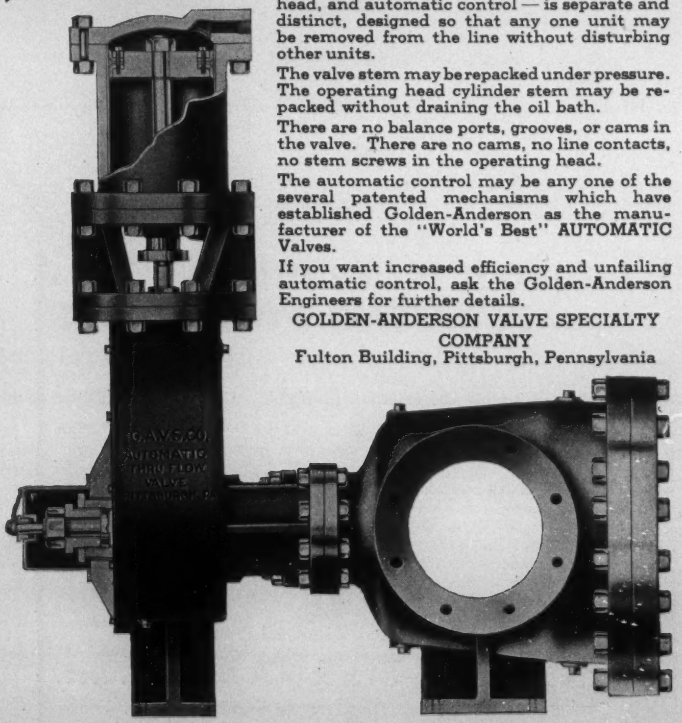
Fulton Building, Pittsburgh, Pennsylvania



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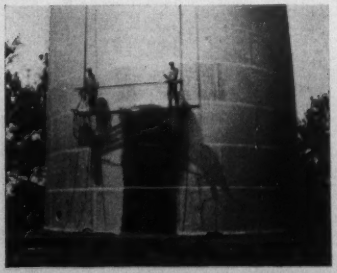

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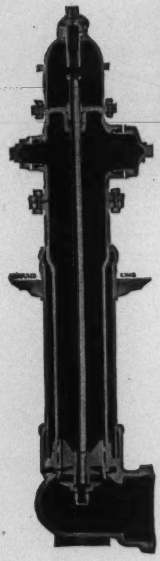

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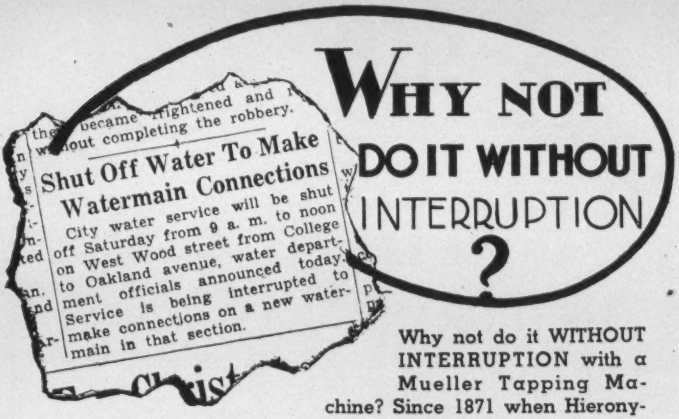
became frightened and
without completing the robbery.
Shut Off Water To Make Watermain Connections
City water service will be shut off Saturday from 9 a. m. to noon on West Wood street from College to Oakland avenue, water department officials announced today. Service is being interrupted to make connections on a new watermain in that section.
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Why not do it WITHOUT INTERRUPTION with a Mueller Tapping Machine? Since 1871 when Hieronymus Mueller perfected the first machine for tapping mains under pressure, this type of shut-down has become needless.

Water Tapping Machine "B"

For tapping water mains and inserting corporation stops ¼" to 1" inclusive. in mains under pressure.

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OF

RED HED

HIGH GRADE BRASS GOODS

FOR

WATERWORKS

IS THE

STRONGEST INSURANCE
AGAINST TROUBLE AND
HIGH MAINTENANCE COSTS

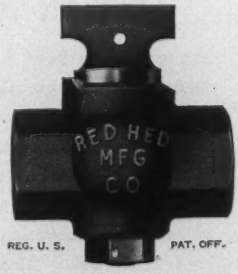

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Fig. 883
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Parallel Seat

—makes ordinary duty easy for a Jenkins

STRENGTH is emphasized in making Jenkins Iron Body Gate Valves for water works service. In both the Solid Wedge and the Double Disc Parallel Seat valves are many superior features that build up strength . . . that make a "Jenkins" capable of handling ordinary duty with ease. The result is longer valve life and less maintenance. This great reserve strength also enables a "Jenkins" to carry-on even if extraordinary strains are encountered.

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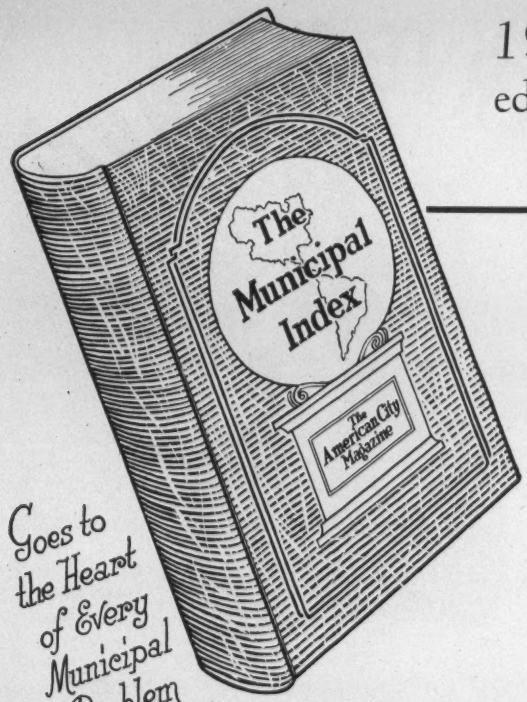
Jenkins Valves

BRONZE—IRON—STEEL SINCE 1864

1937
edition



Goes to the Heart of Every Municipal Problem

THE ANNUAL PURCHASING GUIDE for MUNICIPAL and WATER WORKS OFFICIALS

Published By
THE AMERICAN CITY MAGAZINE
470 Fourth Ave., New York

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1. Low water conditions, or actual shortage of raw water.

2. Excessive algae and other organic growths in raw water

supplies, resulting in excessive tastes and odors.

3. Reduced turbidity in the water supply, making coagulation difficult and shortening filter runs.

4. Greater increased consumer demand.

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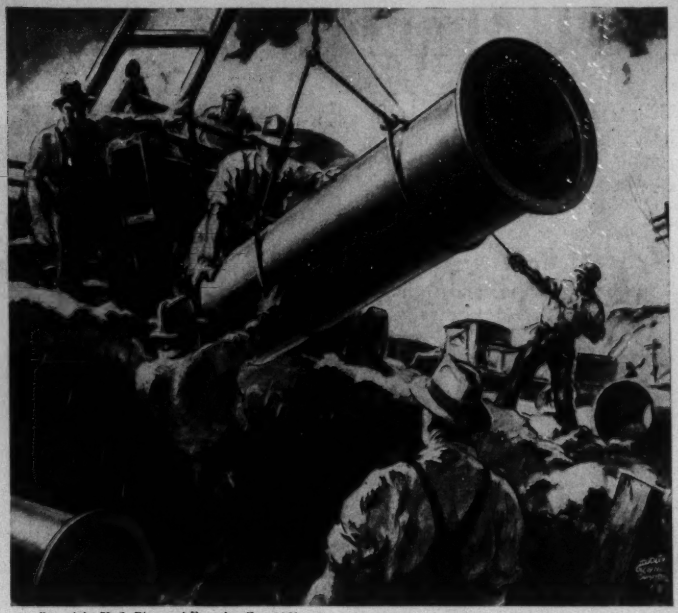

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U.S.
cast iron
PIPE

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U.S. Mechanical Joint Pipe
U. S. Flexible Joint Pipe
U. S. Threaded Cast Iron Pipe
U. S. Ni-Resist Cast Iron Pipe
U. S. Cast Iron Culverts
U. S. Cast Iron Roof Plates
Alloy and Gray Iron Castings

The Journal of the New England Water Works Association

is a quarterly publication, containing the papers read at the meetings, together with reports of the discussions. Many of the contributions are from writers of the highest standing in their profession. It affords a convenient medium for the interchange of information and experience between the members, who are so widely separated as to find frequent meetings an impossibility. Its success has more than met the expectation of its projectors; there is a large and increasing demand for its issues, and every addition to its subscription list is a material aid in extending its field of usefulness.

All members of the Association receive the JOURNAL for two dollars per annum which sum is included in their annual dues; to all others the subscription is four dollars per annum.

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THE attention of parties dealing in goods used by Water Departments is called to the JOURNAL OF THE NEW ENGLAND WATER WORKS ASSOCIATION as an advertising medium.

Its subscribers include the principal WATER WORKS ENGINEERS and CONTRACTORS in the United States. The paid circulation is **1100** COPIES.

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